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24.1 INTRODUCTION

Steel girders are recommended due to depth of section considerations for short span structures and for their economy on longer span structures in comparison to other materials or structure types.

(1) Types of Steel Girder Structures

This Chapter considers the following common types of steel girder structures:

1. Plate Girder
2. Rolled Girder
3. Box Girder

The plate girder structure is selected over the rolled girder structure when longer spans or versatility are required. Generally rolled girders are used for web depths less than 36" (900 mm) on short span structures of 80' (24 m) or less. When rolled girder sections are detailed, a note should be given on the plans allowing the option of using the lighter, fabricated plate girder sections.

(2) Structural Action of Steel Girder Structures

Box, rolled beam and plate girder bridges are primarily flexural structures which carry their loads by bending between the supports. The degree of continuity of the steel girders over their intermediate supports determines the structural action within the steel bridge. The main types of structural action are as follows:

1. Simply Supported Structures
2. Multiple Span Continuous Structures
3. Multiple Span Continuous-Hinged Structures

Simply supported structures are generally used for single, short span structures. Multiple span steel girder structures are designed as continuous spans. When the overall length of the continuous structure exceeds approximately 670' (200 m), a transverse expansion joint is provided by employing girder hinges and a modular watertight expansion device.

The 670’ limitation is based on the abutments having expansion bearings and a pier or piers near the center of the continuous segment having fixed bearings. When one abutment has fixed bearings see Table 12.1 in Chapter 12 for the limitation on the length of a continuous segment.

24.2 MATERIALS

Structural steels currently used conform to ASTM A709M Specifications designated Grade 36 (250), Grade 50 (345), and Grade 50W (345W). AASHTO Specifications give the necessary design information for each grade of steel. Steel girders are economically designed by using high strength steels. In the past high strength steel flanges were recommended in combination with lower strength webs. Current data indicates that "hybrid" girder designs do not necessarily provide the most economical costs. Complete girder designs with Grade 50 (345) steel provide the design advantages of a higher strength plate at a nominal price differential. For unpainted structures over stream crossings, Grade 50W (345W) weathering steel is recommended throughout.

Cracks have been observed in steel girders due to fabrication, fatigue, brittle fractures and stress corrosion. To insure against structural failure the material is tested for plane-strain fracture toughness. As a result of past experience, the Charpy V-notch test is currently required on all grades of steel used for girders.

Refer to Bridge Manual, Chapter 9 - MATERIALS, for additional information.

(1) Bars and Plates

Bars and Plates are grouped under flat rolled steel products that are designated by size as follows:

Bars: 8" (200 mm) or less in width

Plates: Over 8" (200 mm) in width

AASHTO Specifications allow a minimum thickness of 5/16" (8 mm) for structural steel members. Current policy is to employ a minimum thickness of 7/16" (11 mm) for primary members and a minimum 3/8" (10 mm) for secondary structural steel members. The maximum plate width rolled is approximately 200" (5080 mm) at a limited number of mills. Optional splices are permitted on plates which are detailed over 60' (18 m) long. Refer to the latest steel product catalogs for steel sections and rolled stock availability.

(2) Rolled Sections

A wide variety of structural steel shapes are produced by steel manufacturers. Design and detail information is available in the AISC Manual of Steel Construction and information on previously rolled shapes is given in AISC Iron and Steel Beams 1873 to 1952. Refer to the latest steel product catalogs for availability and cost, as some shapes are not readily available and their use could cause costly construction delays.

(3) Threaded Fasteners

Steel connections are made with high strength bolts conforming to ASTM designations A325 and A490. Galvanized A490 bolts can not be substituted for A325 bolts; if A490 bolts are galvanized failure may occur due to hydrogen embrittlement. ASTM specifications limit galvanizing to A325 or lower strength fasteners. All bolts for a given project should be from the same location and manufacturer.

High strength pin bolts may be used as an alternate to A325 bolts. The shank and head of the high strength steel pin bolt and the collar fasteners shall meet the chemical composition and mechanical property requirements of ASTM designation A325, Types 1, 2 or 3.

Friction type connections are used on bridges since the connections are subject to stress reversals and bolt slippage is undesirable. High strength bolts in friction type connections are not designed for fatigue. The allowable unit stresses, minimum spacing and edge distance as given in AASHTO are used in designing and detailing the required number of bolts. A490 bolts shall not be used in tension connections due to their low fatigue strength. Generally, A325 bolts are used for steel connections unless the higher strength A490 bolt is warranted. If at all possible, avoid specifying A490, Type 3 bolts on plans for unpainted structures. All bolt threads should be clean and lubricated with oil or wax prior to tightening.

A. Bolted Connections

1. All field connections are made with 3/4" (M20) high strength bolts unless noted or shown otherwise.
2. Holes for bolted connections shall not be more than 1/16" (2 mm) greater than the nominal bolt diameter.
3. Faying surfaces of friction type connections are blast cleaned and free from all foreign material. Note that AASHTO Specifications allow various design shear stresses depending on surface condition of bolted parts. Higher design shear stresses are allowed on blast cleaned and/or inorganic zinc-rich painted surfaces.
4. Bolts are installed with a flat, smooth, hardened circular washer under the nut or bolt head, whichever element is turned in tightening the connection.
5. A smooth, hardened, bevel washer is used where bolted parts in contact exceed a 1 to 20 maximum slope.
6. Where clearance is required, washers are clipped on one side to a point not closer than seven-eighths of the bolt diameter from the center of the washer.
7. After all bolts in the connections are installed, each fastener shall be tightened equal to the proof load for the given bolt diameter as specified by ASTM. A490 bolt and galvanized A325 bolts shall not be reused.

Retightening previously tightened bolts which may have been loosened by tightening of adjacent bolts is not considered a reuse.

B. Bolt Weight and Length

Under current specifications the weights of nuts, heads, washers, and the stick through length of high strength bolts are added to the structural steel weights. These weights, as given by AASHTO, are shown in Table 24.1

TABLE 24.1

The length of high strength bolts is determined by adding to the grip (the total thickness of the connected material) the value in Table 24.1 for the given bolt diameter.

Bolt Diameter	Weight of 100 Bolts Pounds (kg)	For Length Add to Grip in.	(mm)
3/4" (M20)	52.4 (23.8)	1 3/16	30
7/8" (M22)	80.4 (36.5)	1 5/16	35
1" (M24)	116.7 (52.9)	1 9/16	40
1 1/8" (M27)	165.1 (75.0)	1 11/16	43
1 1/4" (M30)	212.0 (96.2)	1 13/16	46

These values are to provide for the inclusion of one circular washer, a heavy nut, and adequate stick-through at the end of the bolt. For a beveled washer used in place of circular washer, add an additional 1/8" (3 mm). The length is rounded up to the next 1/4" (5 mm) increment for bolt lengths up to 5" (125 mm) and to the next 1/2" (13 mm) increment for bolt lengths over 5".

24.3 DESIGN SPECIFICATION AND DATA

(1) Specifications

Refer to the design and construction related materials as presented in the following specifications:

American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

Standard Specifications for Welding of Structural Steel Highway Bridges.

American Institute of Steel Construction Manual of Steel Construction.

State of Wisconsin Standard Specifications for Road and Bridge Construction.

(2) Allowable Stresses

The allowable stresses are given in the AASHTO Specifications. The basic ultimate stresses for the more common structural components used on bridges is given in Chapter 9.0 - MATERIALS.

(3) Design Aids

Refer to Standard 24.1 for recommended girder spacing for a given roadway width. Given the span length, the preliminary steel girder web depth is determined by referring to Table 24.2. Recommended web depths are given for parallel flanged steel girders. The girder spacings and web depths were determined from an economic study, deflection criteria, and load carrying capacity of girders.

From a known girder spacing, the effective span is computed as shown on Figure 17.1. From the effective span, the slab depth and required slab reinforcement are determined from Tables as well as the additional slab reinforcement required due to slab overhang.

(4) References for Horizontally Curved Structures

Standard 24.10 shows the method for laying out steel girders on horizontally curved bridges. When the radius of the curve is 830 feet (250 m) or greater, the girders are fabricated by kinking them at the field splice locations. If the radius of the curve is less, the girders are fabricated along the curve.

PARALLEL FLANGE GIRDER RECOMMENDED DEPTHS

(For 2-Span Bridges with Equal Span Lengths)

10' (3 m) (Girder Spacing 9” (225 mm) Deck)				12' (3.6 m) Girder Spacing 10” (250 mm) Deck)			
Span Lengths		Web Depth		Span Lengths		Web Depth	
m	(ft)	mm	(In)	m	(ft)	mm	(In)
27-35	(90-115)	1200	(48)	27-31	(90-103)	1200	(48)
35-39	(116-131)	1350	(54)	31-36	(104-119)	1350	(54)
39-42	(132-140)	1500	(60)	36-38	(120-127)	1500	(60)
42-45	(141-149)	1650	(66)	38-41	(128-135)	1650	(66)
45-49	(150-163)	1800	(72)	41-44	(136-146)	1800	(72)
49-51	(164-171)	1950	(78)	44-46	(147-153)	1950	(78)
51-54	(172-180)	2100	(84)	46-49	(154-163)	2100	(84)
54-57	(181-190)	2250	(90)	49-51	(164-170)	2250	(90)
57-60	(191-199)	2400	(96)	51-53	(171-177)	2400	(96)
60-62	(200-207)	2550	(102)	53-55	(178-184)	2550	(102)
62-65	(208-215)	2700	(108)	55-58	(185-192)	2700	(108)

TABLE 24.2

Criteria used to develop table:

- Girder designed by Load Factor Design Method using HS25 (MS22.5) loading.
- Fatigue check using HS20 loading.
- Deflection check using HS25 based on limiting (LL) deflection to L/1200.
- Standard Permit Vehicle capacity must be 190 kips (845 kN) or greater, using Load Factor Analysis with (1.3 DL + 1.3 LL) and single lane distribution.
- Type A709 Grade 50 (345) steel used for girders.

- Maximum flange plate thickness used was 2 1/4" (55 mm)

- Recommended ratio of girder flange width to total girder depth (b/d) is given as 0.3 for shallow girders to about 0.2 for deep girders. (Charles G. Salmon and John E. Johnson, Steel Structures, Harper and Ron, Publishers, 1980).

| The Approximate Method of Design developed by USS Corporation is an accepted approach for horizontal curves with a radius greater than 585 feet (175 meters). This method is outlined in publications by the Corporation and stored in the Bridge Library. Also, the designer may refer to "Tentative Design Specifications for Horizontally Curved Highway Bridges" prepared for the FHWA under Research Project HPR 2-(111). These tentative specifications should be used along with an allowable analysis program.

| The computer program CBRIDGE, developed at Syracuse University, was purchased in 2001 and is available for the analysis and design of curved or straight girder highway bridges. CBRIDGE utilizes a three dimensional method of analysis and designs by LFD. Lateral bracing may or may not be considered at the option of the user.

24.4 DESIGN CONSIDERATIONS

| Steel girder structures are analyzed and designed by the Load Factor Design Method. AASHTO Specifications provide all the details of designing simple and continuous steel girders for various span lengths by Load Factor Design Method. It is a method of proportioning structural members for multiples of design service loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings, and to the control of live load deflections under service loadings. Three theoretical load | levels are employed: Maximum Design Load, Overload, and Service Load. Designers should use SIMON Computer Program incorporating the requirements of deflection and Load Factor Design.

The procedures for dead load distribution, lateral distribution of live load, computations of reactions, shears, moments and deflections, determination of effective slab widths, section properties (except for plastic section modulus and related properties) and stresses in composite sections are the same for Service and Load Factor Design.

(1) Distribution of Loads

Refer to AASHTO Specifications.

A. Dead Load

1. The uniform dead load of the slab is determined using the concrete unit weight and simple beam distribution. No adjustment in weight is made for bar steel reinforcement.

2. The weight of the concrete haunch is determined by estimating the haunch depth at 1-1/4" (30 mm) and the width equal to the largest top flange of the supporting member.

3. The weight of steel beams and girders is determined from the AISC Manual of Steel Construction. Haunched webs of plate girders are converted to an equivalent uniform partial dead load.

4. The weight of secondary steel members such as bracing, shear studs, and stiffeners is estimated at 30 plf (440 N/m) for interior girders and 20 plf (290 N/m) for exterior girders.

| 5. A dead load of 20 psf (1.0 kN/m^2) carried by the composite section is added to account for a future wearing surface.

6. The AASHTO Specification allows the weight of sidewalks, curbs, parapets, medians, railings and other dead loads placed after the slab has cured to be equally distributed to all members. However, in the Bridge Office, a simple beam distribution is generally used.

The dead load carried by the exterior girder consists of those loads lying outside the centerline of the exterior girder and a simple beam distribution of those loads lying between the center lines of the exterior and interior girders. AASHTO specifies that the effect of creep is to be considered in the design of composite girders which have dead loads acting on the composite sections. Stress and horizontal shears produced by these dead loads are computed for a value of n or 3n, whichever gives the higher stresses and shears.

Dead load deflections of all girders are assumed equal to a typical interior girder on a bridge with a standard curb and/or parapet on each side. If a sidewalk is located on one or both sides of the bridge, the deflections for interior and exterior girders are computed separately. Distribution of the dead loads is the same as for design of the girder. Total deflections for concrete are computed to the nearest 1/8" (3 mm). Values are computed at span quarter points and all field splice points.

B. Traffic Live Load

1. The HS25 (MS22.5) live load bending moment for each interior beam or girder is determined by applying S/5.5 (S/1.68) wheel loads and S/11 (S/3.36)lane load to the member. S is the girder spacing in feet (meters).
| Check fatigue using HS20 loading.

1A. The Standard Permit Capacity must be 190 Kips or greater with load factors (1.3 DL + 1.3 LL) and single lane distribution.

2. The live load bending moment for each exterior beam or girder is determined by applying the larger wheel load distribution computed by either a simple beam distribution or the distribution indicated below.

| S/5.5 (S/1.68) where S $\leq$ 6' (1.8 m)
S/(4 + 0.25S) [S/(1.22 + .25S)] where S > 6' (1.8 m) but < 14'(4.27 m)
Simple beam distribution for S $\geq$14' (4.27 m) where S is the spacing between exterior and interior adjacent girders in feet (meters)

The simple beam distribution is determined by placing the outside wheel at 2' (600 mm) from the face of the curb. The wheels of the truck are then spaced 6' (1.8 m) apart and the distance to the wheel load of an adjacent truck is 4' (1.2 m).

3. The live load shears and end reaction of interior and exterior members at points of support are determined by applying a simple beam distribution to the first set of wheels at the support and next applying the distribution factor prescribed for moment for subsequent wheel loads. For an exterior member, the simple beam wheel load distribution is usually less than the distribution factor specified for moment. The AASHTO specifications permit this type of wheel loading for end shears and reactions.

4. The intermediate live load shears within the span of interior and exterior members are determined by applying the wheel load distribution factor for moment to all sets of wheel loads.

5. The live load moments, shears, and deflections are computed based on composite section properties of the girder. Negative composite action is not used since shear studs are not detailed in the negative moment regions of the span.

C. Pedestrian Live Load

Sidewalk floors, stringers and their immediate supports and bridges for pedestrians and/or bicycle traffic are designed for a live load of 85 psf (4.25 kN/m^2) of sidewalk area. Girders, trusses, arches and other members shall be designed for the following sidewalk live loads:

Spans 0 to 25 feet .... 85 psf (4.25 kN/m^2)
Spans 26 to 100 feet .... 60 psf (3.0 kN/m^2)
Spans over 100 feet according to AASHTO

The sidewalk live load is converted to weight per linear foot by multiplying by the width of sidewalk. When the exterior beam or girder supports the sidewalk live load as well as the traffic live load and impact, the allowable design stress is increased by 25 percent for the combination of dead load, sidewalk live load and traffic live load plus impact, providing that the exterior girder is not of less capacity than the interior girder. The chain link fencing is assumed to carry no wind loading for design computations.

D. Temperature

Steel girder bridges are designed for a coefficient of linear expansion equal to .0000065/F (.0000117/C) at a temperature range from -30 to 120°F (-35 to 50°C). Refer to Chapter 28 - Expansion Devices for expansion joint requirements.

E. Wind

The following wind loads for a wind velocity of 100 mph (160 km/h) are subject to the loading combinations and percentage increase in basic allowable stress as permitted by AASHTO specifications.

1. For steel beams and girders a uniformly distributed wind load of 50 psf (2.5 kN/m^2) but not less than 300 plf (4400 N/m), is applied horizontally at right angles to the elevation view of the superstructure.

2. A wind load of 100 plf (1460 N/m) is applied at right angles and 6' (1.8 m) above the deck as a wind load on a moving live load.

3. An overturning wind load of 20 psf (1.0 kN/m^2) on the total bridge width is applied at the quarter point of the transverse bridge width.

The wind loads described above are resisted by the concrete slab and a system of lateral bracing or diaphragms. The wind loading is transferred to the substructure units through the bearings.

(2) Minimum Depth to Span Ratio

For composite girders the ratio of overall depth of girder (concrete slab plus steel girder) to the length of span preferably should not be less than 1/25, and the ratio of the depth of the steel girder to length of span preferably should not be less than 1/30.

(3) Live Load Deflections

A. Allowable Deflection

AASHTO specifies the deflection criteria for steel beams and girders of simple or continuous spans. This is modified by the Bridge Office limiting maximum allowable deflection to (L/1200), including spans having hinges using the design live loads for HS20 Lane or Truck loading with Impact. Limiting live load deflections is an effort to provide a structure having greater stiffness and thereby increasing deck durability by reducing concrete cracking. For steel bridges having a hinge, the allowable deflection at the hinge (L/375) may be exceeded as long as the deflection within the span does not exceed the limits stated above.

B. Actual Deflection

The distribution factor for computing live load deflection is not always the same as the moment distribution factor because it is assumed that all the beams or girders act together and have an equal deflection. For example, a 40' (12 m) width of bridge between curbs with 4 supporting girders carries a maximum of 3 trucks or lanes with a 90 percent reduction in load intensity. The wheel load distribution is as follows:

$$\frac{\text{(3) Trucks (2 Wheels/Truck) (90\%)}}{\text{4 Girders}} = 1.35 \frac{\text{Wheels}}{\text{Girder}}$$

(4) Uplift and Pouring Diagram

Permanent hold down devices are used to attach the superstructure to the substructure at the bearing when any combination of loading with a 100 percent increase in live load plus impact produces uplift. Also, permanent hold down devices are required on alternate girders that cross over streams with less than 2' (600 mm) clearance for a 100 year flood where expansion bearings are used. These devices are required to prevent the girder from moving off the bearings during extreme flood conditions.

Uplift generally occurs under live loading on continuous spans when the span ratio is greater than 1 to 1.75. Under extreme span ratios the structure may be in uplift for dead load. When this occurs it is necessary to jack the girders upward at the bearings and insert shim plates to produce a downward dead load reaction. The use of simple spans or hinged continuous spans is also considered for this case.

On two span bridges of unequal span lengths, the slab is poured in the longer span first. Cracking of the concrete slab in the positive moment region has occurred on bridges with extreme span ratios when the opposite pouring sequence has been followed. When the span exceeds 120' (36 m), consider some method to control positive cracking such as limited pouring time, the use of retarders, and sequence of placing.

On multiple span structures determine a pouring sequence that causes the least structure deflections and permits a reasonable construction sequence. Refer to Standard 24.11 for concrete slab pouring requirements. Temporary hold down devices are placed at the ends of continuous girders where the slab pour ends if uplift does not occur from dead load and/or live load. The temporary hold down devices prevent uplift and unseating of the girders at the bearings during the pouring sequence.

Standard hold down devices having a capacity of 20 kips (88.9 kN) are attached symmetrically to alternate girders or to all the girders as required. Hold down devices are designed by considering line bearing acting on a pin. Refer to Standard 27.6 for permanent and temporary hold down details. To compute uplift, a shear influence line is first obtained. Next the wheel load distribution factor is determined in the same manner as for live load deflection. The number of loaded lanes is based on the width of the bridge between curbs. The live load plus impact is uniformly distributed to all the girders and is reduced in multiple lane loadings. The truck or lane live load is increased 100 percent and applied to the shear influence line to produce maximum uplift. The allowance for future wearing surface should not be included in uplift computations when this additional dead load increases the end reaction.

(5) Bracing

All bracing systems must be attached to the main girder by bolted connections. Connections within the system are also bolted.

A. Intermediate Diaphragms and Cross Frames

Diaphragms or cross frames are required at each support and throughout the span at 25' (7.6 m) maximum centers in all bays as specified by AASHTO. The spacing is adjusted to miss any splice material. The transverse bracing is placed parallel to the skew for angles up to and including 15 degrees and normal to the girders for skew angles greater than 15 degrees. When diaphragms are stepped slightly out of straight through alignment, the girder flanges will experience the greatest torsional stress. Larger steps in diaphragm spacing allow the torsional moment to distribute over a longer girder section. On curved girder structures, the diaphragms are placed straight through on radial lines to minimize the effects of torsion since the diaphragms or cross frames are analyzed as primary load carrying members.

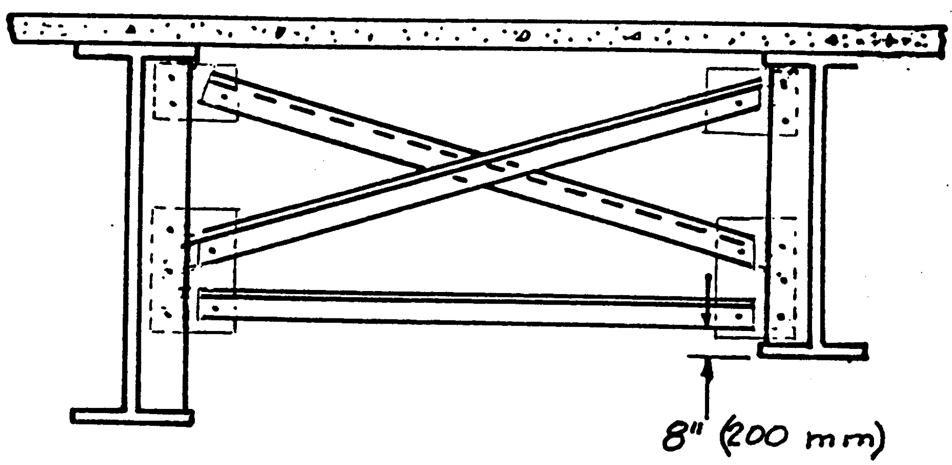
Diaphragm details and dimensions are given on Standards 24.3 and 24.6. Diaphragms carry moment and tensile stresses caused by girder deflections. In the composite slab region, the steel section acts similar to the lower chord of a vierendeel truss and is in tension. A rigidly connected diaphragm resists bending due to girder deflection and tends to distribute the load. It is preferable to place diaphragms at the 0.4 point of the end spans on continuous spans and at the center of interior spans when this can be accomplished without an increase in total number. Also, if practical place diaphragms adjacent to a field splice between the splice and the pier. Bolted diaphragm connections may be used in place of welded diaphragm connections. All cross framing is attached to this main girder via bolted gusset plates.

Cross framing is used for web depths over 48" (1225 mm). The bracing consists of two diagonal members connected at their intersection and one bottom chord member. The bottom chord is designed as a secondary compression member. The diagonals are designed as secondary tension members. The length of a minimum 1/4" (6 mm) fillet weld size is determined for each member based on a minimum of 75 percent of the member strength.

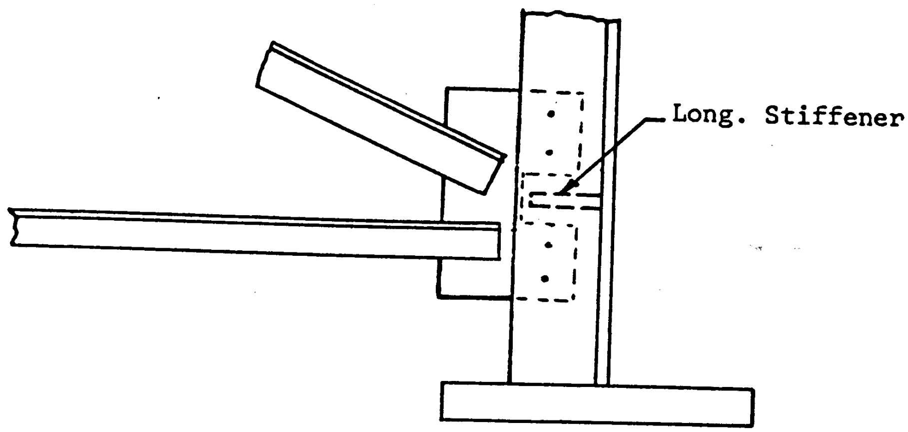
On spans over 200' (60 m) in length the stresses caused by wind load on part of the erected girders without the slab in place may control the size of the members. Construction loads are also considered in determining member size.

If welded connections are used, the members which make up the "X" portion of the bracing are attached with erection bolts which are left in place after welding the member ends. The "X" angles are field welded at these intersections.

On haunched steel girders, the cross framing follows the curvature of the girder. The lower chord is placed at 8" (200 mm) above the higher bottom flange of the adjacent girders. Refer to sketch below:



On girders where longitudinal stiffeners are used, the relative position of the stiffener to the cross frame is checked. When the longitudinal stiffener interferes with the cross frame, cope the gusset plate attached to the vertical stiffener and attach the cross frame to the gusset plates as shown below.



B. End Diaphragms

End diaphragms are placed horizontally along the abutment end of beams or girders and at other points of discontinuity in the structure. Channel sections are generally used for end diaphragms which are designed as simply supported edge beams. The live load moment plus impact is determined by placing one wheel load or two wheel loads 4' (1.2 m) apart and correcting for the skew angle at the center line of the member. Generally, the dead load moment of the overlying slab and diaphragm is insignificant and as such is neglected. End diaphragm details and dimensions are given on Standard 24.4.

End diaphragms are either bolted or welded to gussets attached to the girders at points of discontinuity in the superstructure. The gusset plates are bolted to the girders. The same connection detail is used throughout the structure. The connections are designed for shear only where joined at a web since very little moment is transferred without a flange connection. The connection is designed for the shear plus impact from the wheel live loads.

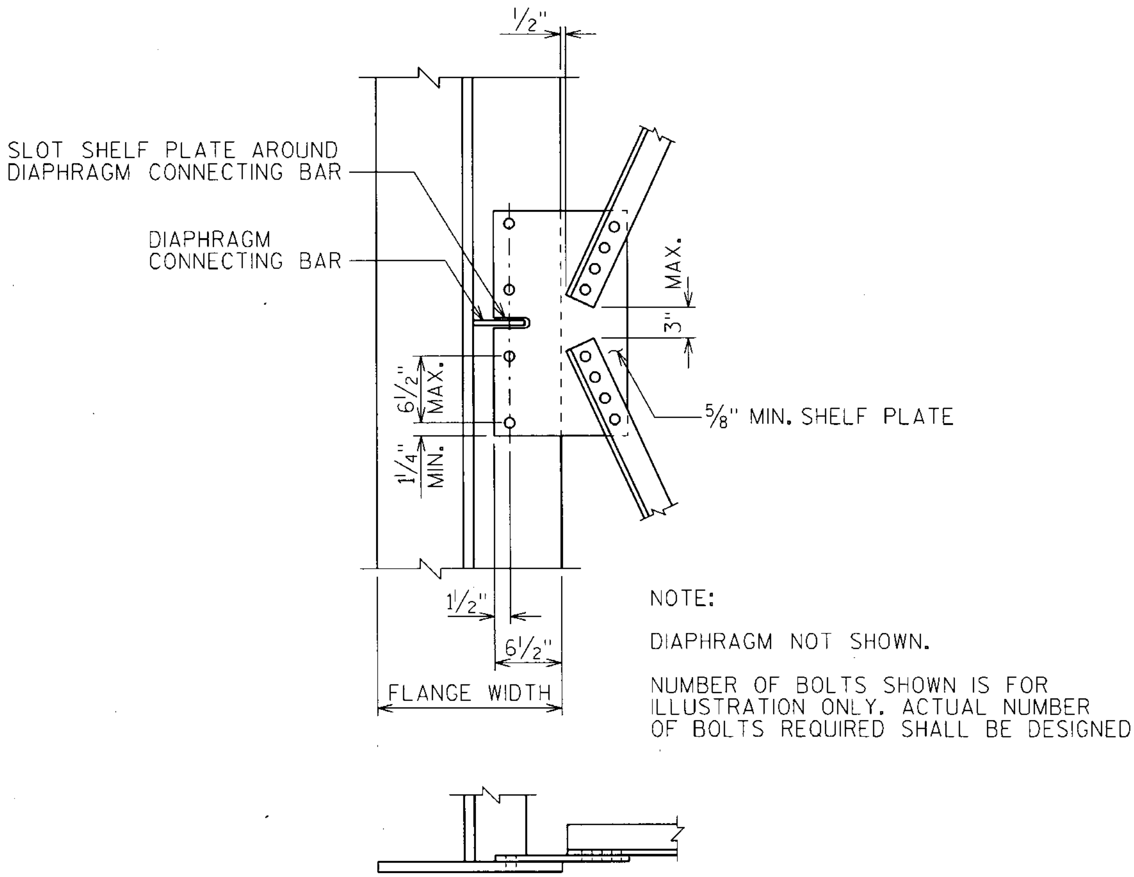
C. Lower Lateral Bracing

Lateral bracing requirements for the bottom flanges are to be investigated as per AASHTO Specifications. BOS practice is to eliminate the need for bracing by either increasing flange sizes or reducing the distance between cross frames. The controlling case for this stress is usually at a beam cutoff point. At cutoff points condition of maximum stress exists with the smallest flange size; here wind loads have the most effect. A case worth examining is the temporary stress that exists in top flanges during construction. These plates which are often only 12" (300 mm) wide can be heavily stressed by wind load. A temporary bracing system placed by the contractor may be in order.

On an adjacent span to one requiring lower lateral bracing, the bracing is extended one or two panel lengths into that span. The lower lateral system is placed in the exterior bays of the bridge and in at least 1/3 of the bays of the bridge. On longer spans the stresses caused by wind load during construction will generally govern the member size.

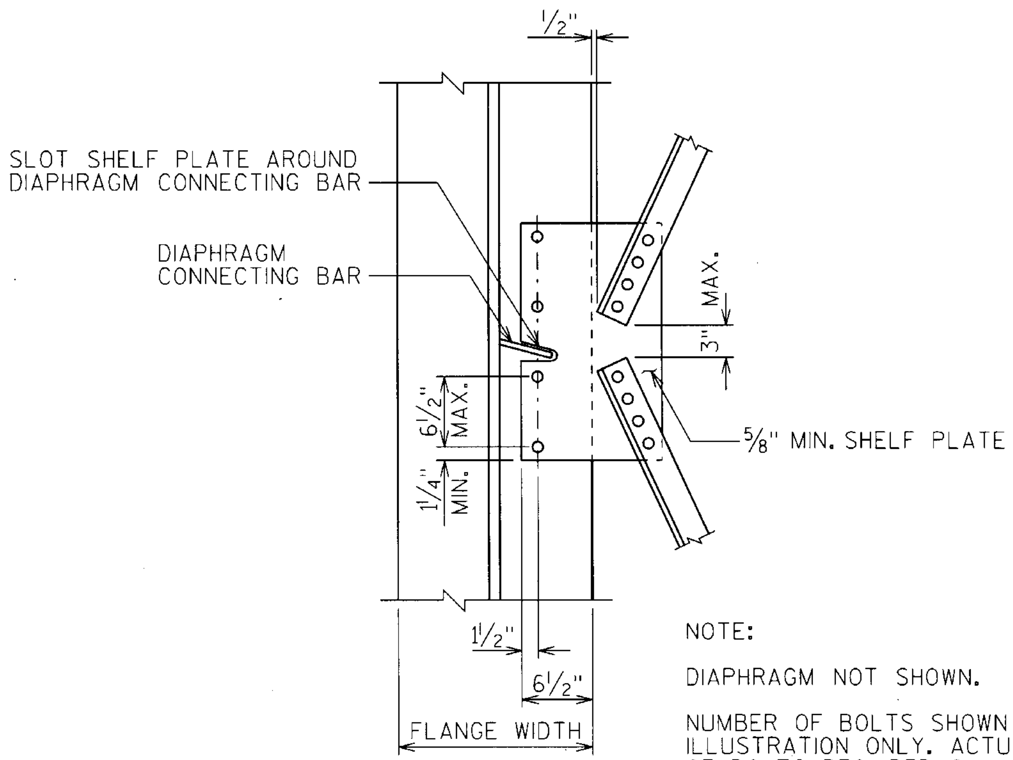
Lateral bracing consists of two diagonals connected at their intersection. Bracing must be designed and is attached to the bottom flange as shown in the Figures. The length of a minimum 1/4" (6 mm) fillet weld size is determined for each diagonal based on 75 percent of the strength of the member. Since the effective length in one plane is half that in the other plane unsymmetrical angles are used.

For curved girders MDX and DESCUSS II do not consider lower laterals in the analysis and therefore they are not required if the design is developed using one of these programs. C-BRIDGE at the option of the designer will consider lower laterals in the analysis and then they would be sized based on the stresses that the program computes. Also our curved girders do not have extremely long span lengths and the curvature of the girders forms an arch which is usually capable of resisting the wind forces prior to placing the slab.



LOWER LATERAL CONNECTION DETAIL IN SPAN, SKEW > 15° OR 0°

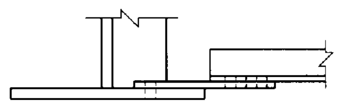
(BETWEEN BEARINGS AT PIERS AND ABUTMENTS)



NOTE:

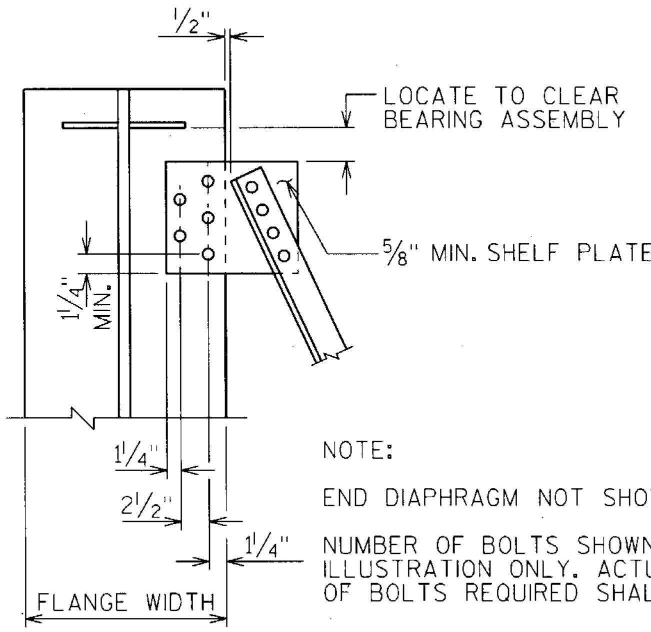
DIAPHRAGM NOT SHOWN.

NUMBER OF BOLTS SHOWN IS FOR ILLUSTRATION ONLY. ACTUAL NUMBER OF BOLTS REQUIRED SHALL BE DESIGNED.



LOWER LATERAL CONNECTION DETAIL IN SPAN, SKEW $\leq$ 15°

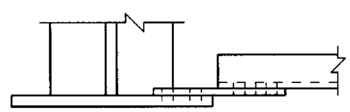
(BETWEEN BEARINGS AT PIERS AND ABUTMENTS)



NOTE:

END DIAPHRAGM NOT SHOWN.

NUMBER OF BOLTS SHOWN IS FOR ILLUSTRATION ONLY. ACTUAL NUMBER OF BOLTS REQUIRED SHALL BE DESIGNED.



LOWER LATERAL DETAILS AT ABUTMENT AND PIERS

(6) Girder Selection

The exterior girder section is always designed and detailed equal to or larger than the interior girder sections. Ratios of girder depth to length of span which is dependent on type of design are not to exceed values recommended by AASHTO Specifications. The following criteria are used when determining the selection and sizes of girder sections.

A. Rolled Girders

Rolled sections without cover plates are preferred. Cover plates are not recommended due to fatigue considerations and higher fabrication costs. The following guidelines are used for cover plate design and detailing.

1. Cover plate widths are to be less than the flange width minus 25 mm. A larger width cover plate may be used if it is wider than the top flange only in very special cases. Maximum cover plate thickness is two times the thickness of the flange to which it is attached.

2. A partial length welded cover plate is extended beyond its theoretical end and terminated at bolted field splice locations.

3. A shop welded butt splice is considered in place of adding cover plates. Also, a welded plate girder section is considered at piers where a rolled girder section requires heavy cover plates.

B. Plate Girders

1. Maximum change in flange plate thickness is 1" (25 mm) inch and preferably less. The thinner plate is not less than 1/2 the thickness of the thicker flange plate. Plate thicknesses are given in the following increments:

   1/16 inch thru 1” (10, 11, 12, 14, 16, 18, 20, 22 & 25 mm)
   1/8 inch up to 2” (28, 30, 32, 35, 38, 40, 45, 50, 55 & 60 mm)
   1/4 inch above 2” (10 mm increments above 60 mm)

2. Minimum plate size on the top flange of a composite section is variable depending on the depth of web, but not less than 12 x ¾ (300 x 20 mm) for web depths less than or equal to 66” (1675 mm). Thinner plates become wavy and require extra labor costs to straighten within tolerances. For plate girder flange widths, use 1 inch (25 mm) increments. For plate girder web depths, use 1 inch (25 mm) increments. Changes in plate widths or depths are to follow recommended standard transition distances and/or radii.

The minimum size flange plates of 16" x 1 1/2" (400 x 40 mm) at the point of maximum negative moment and 16" x 1" (400 x 25 mm) at the point of maximum positive moment are recommended for use at plate girders. The use of a minimum flange width on plate girders is necessary to maintain adequate stiffness in the girder so it can be fabricated, transported, and erected. Deeper web plates with small flanges may use less steel but create problems during fabrication and construction. However, flange sizes on plate girders with web depths 48" (1225 mm) or less may be smaller.

3. Flange plate sizes are detailed based on recommended maximum span lengths given in Table 24.2 for parallel flanged girders. The most economical girder is generally the one having the least total weight, but is determined by comparing material costs and welding costs for added stiffener details. Plates over 60' (18 m) are difficult to obtain and butt splices are detailed to limit flange plates to these lengths or less. It is better to detail more flange butt splices than required and leave the decision to utilize them up to the fabricator. All butt splices are made optional to the extent of available lengths and payment is based on the plate sizes shown on the plans. Since the mills no longer roll plate widths less than 48" (1.2 m), detail flange plates to the same width and vary the thicknesses. This allows easier fabrication when cutting plate widths.

4. Minimum web thickness is 7/16" (11 mm) for girder depths less than or equal to 60" (1500 mm). An economical web thickness usually has a few transverse stiffeners. Refer to Article 24.10 for transverse stiffener requirements. Due to fatigue problems, use of longitudinal stiffeners in built up members are not encouraged.

(7) Welding

Welding design details shall conform to current requirements of AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges. Refer to the AISC Engineering Journal (1973) for a commentary on highly restrained welded connections and lamellar tearing of materials. Due to a number of failures in electroslag welds in 1976-77, this weld procedure is no longer permitted.

Weld details are not shown on the plans, but are indicated by using standard symbols as given on Chart 24.1. Weld sizes are based on the size required due to stress or the minimum size for plate thicknesses being connected. The strength of a fillet weld equals the fillet size times the allowable basic shear stress times (0.707) for equal leg weld sizes. The effective or net section is taken as the shortest distance from the root to the face of the weld.

The following formula is used to compute the weld size for connecting the flange of a plate girder to the web:

$$R = \frac{M' - M}{ad} = \frac{C' - C}{a}$$

where
- R = Rate of change in flange stress, kips/inch
- M' & M = Moments at sections, inch-kips
- C' & C = Force in flange at the sections, kips
- a = Distance between sections of M' & M, inches
- d = Effective web depth, inches

Knowing the rate of change in flange stress, the minimum size weld required to carry this force is chosen and compared to the minimum weld size required. Generally the minimum weld size governs due to the material thicknesses employed.

The following data is used to determine minimum fillet weld size based on the thickness of the thicker part joined.

Fillet Weld Size	Thickness of Thicker Part Joined
3/16" (5 mm)	to 1/2" (12 mm)
1/4" (6 mm)	over 1/2-3/4" (12 to 18 mm)
5/16" (8 mm)	over 3/4-1 1/2" (18 to 38 mm)
3/8" (10 mm)	over 1 1/2-2 1/4" (38 to 55 mm)
1/2" (13 mm)	over 2 1/4-6" (55 to 150 mm)

The fillet weld size is not required to exceed the thickness of the thinner part joined. Refer to current AASHTO specifications for minimum effective fillet weld length and end return requirements.

Bevel welds are used to develop the full strength of a member. Specify the depth of the bevel as 1/8" (3 mm) less than plate thickness due to weld burn through of at least that much. Since this type of weld develops full member strength, a strength check is not required.

(8) Camber and Blocking

When straight girder sections between splice joints are erected, final girder elevations usually vary in height between the girder and roadway elevations due to dead load deflections and vertical curves. Since a constant slab thickness is detailed, a concrete haunch between the girder and slab is used to adjust these variations. If these variations exceed 3/4" (20 mm), the girder is cambered to reduce the variation of thickness in the haunch. Straight line chords between splice points are sometimes used to create satisfactory camber.

Welded girders are cambered by cutting the web plates to a desired curvature. During fabrication all web plates are cut to size since rolled plates received from the mill are not straight. There is a problem in fabricating girders that have specified cambers less than 3/4" (20 mm) so they are not detailed.

Rolled sections are cambered by the application of heat in order that less camber than recommended by AISC specification may be used. The concrete haunch is used to control the remaining thickness variations.

A blocking diagram is given for all continuous steel girder bridges on vertical curve. Refer to Standard 24.9 for blocking and slab haunch details. Blocking heights to the nearest 1/8" (3 mm) are given at all bearings, field splices, and shop splice points. The blocking dimensions are from a horizontal base line passing through the lower end of the girder at center line of bearing.

In a table show the top of steel elevations after erection at each field splice and centerline of all bearings. Total dead load and concrete only deflections to nearest 1/8" at tenth points are to be shown on a deflection diagram.

<u>Note</u>: The plans are detailed for horizontal distances. The fabricator must detail all plates to the erected position considering dead loads. Structure erection considerations are 3-dimensional considering slope lengths and member rotation for member end cuts.

(9) Expansion Hinges

The "Expansion Hinge" as shown on Standard 24.8 is used where pin and hanger details were previously used. It is more redundant and, if necessary, the bearings can easily be replaced. The recommended distance between joints is given on Standard 24.8. These limits are based on surveys of transverse deck cracking on steel bridges in Wisconsin over 500' (150 m) long. Continuous steel units greater than these limits generally had severe transverse deck cracking.

24.5 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

Current AASHTO specifications for Structural Steel Design contain major revisions to previous fatigue design-detail guidelines and requires material impact testing for fracture toughness. These revisions and additions are the results of performance evaluations over the past decade on existing highways and bridges under the action of repetitive vehicle loading.

The direct application of fatigue specifications to main load carrying members has generally been apparent to most bridge designers. As a result main members have been designed with the appropriate details. However, fatigue considerations in the design of secondary members and connections has not always been so obvious. Many of these members interact with main members and receive more numerous cycles of load at a higher level of stress range than assumed. This accounts for most of the fatigue problems surfacing in recent years as cracking initiated by secondary members.

(1) Fatigue Strength

The main factors governing fatigue strength are the applied stress, the number of loading cycles, and the type of detail. The designer has the option of either limiting the stress range to acceptable levels or choosing details which limit the severity of the stress concentrations.

Details involving connections that experience fatigue crack growth from weld toes and weld ends where there is high stress concentration provide the lowest allowable stress range. This results for both fillet and groove welded details. Details which serve the intended function and provide the highest fatigue strength are recommended.

Generally, details involving failure from internal discontinuities such as porosity, slag inclusion, cold laps and other comparable conditions, will have a high allowable stress range. This is primarily due to the fact that geometrical stress concentrations at such discontinuities do not exist other than the effect of the discontinuity itself.

AASHTO Specifications provide the designer with six basic design range categories for redundant and non-redundant load path structures. The stress range category is selected based on the highway type and the detail employed. The designer may wish to make reference to "Bridge Fatigue Guide Design and Details" by John W. Fisher.

(2) Charpy V-Notch Impact Requirements

Recognizing the need to prevent brittle fracture failures of main load carrying structural components, AASHTO Material Specifications adopted provisions for Charpy V-Notch impact testing in 1974. Impact testing offers an important measure of material quality, particularly its ductility. Brittleness is detected prior to placing the material in service to prevent member service failures. Wisconsin Standard Specifications for Road and Bridge Construction require Charpy V-Notch tests on all girder flange and web plates, flange splice plates, hanger bars, links, rolled beams and flange cover plates. Special provisions require higher Charpy V-Notch values for non-redundant structure types.

(3) Non-Redundant Type Structures

Previous AASHTO fatigue and fracture toughness provisions provided satisfactory fracture control for multi-girder structures when employed with good fabrication and inspection practices. However, concern existed that some additional factor of safety against the possibility of brittle fracture should be provided in the design of non-redundant type structures such as single box girders, two plate girder or truss systems where failure of a single element could cause collapse of the structure. A case in point was the collapse of the Point Pleasant Bridge over the Ohio River.

Primary factors controlling the susceptibility of non-redundant structures to brittle fracture are the material toughness, flaw size and stress level. One of the most effective methods of reducing brittle fracture is lowering the stress range imposed on the member. In 1977, AASHTO Specifications provided an increased safety factor for non-redundant members by requiring a shift of one range of loading cycles for fatigue design with corresponding reduction of stress range for critical stress categories. The separate tables for redundant and non-redundant load path structures clearly indicates the need for special cases in the AASHTO Specification. The restrictive ranges for certain categories will require the designer to investigate the use of details which do not fall in critical stress categories or induce brittle fracture.

24.6 DESIGN APPROACH - STEPS IN DESIGN

(1) The following steps are taken in determining the Girder or Beam Spacing and the Slab Thickness:

The girder spacing (number of girders) for a structure is determined by considering the desirable girder depth and the span lengths. Refer to Section 24.3(3) for design aids. For typical roadways the girder spacing is shown in the Standard 24.1. For other roadways it is usually more economical to use fewer girders with a wider spacing and a thicker slab.

The slab overhang on exterior girders is limited to 3'-7" (1100 mm) measured from the girder centerline to the edge of slab. The overhang is limited to prevent rotation and bending of the web during construction caused by the forming brackets.

Check if a thinner slab and the same number of members can be used by slightly reducing the spacing. Refer to Tables 17.1 and 17.2 for effective girder span versus slab thickness.

(2) The following steps are taken in estimating the Size of Beams and Girders:

Use design tables and similar structures previously designed in estimating the size of the members. Refer to Table 24.2 for recommended girder depths for a given girder spacing and span length.

Use the same width of bottom flange throughout the bridge whenever possible. High bearing reactions at the piers of continuous girders may govern the width of the bottom flange.

Strive for a balanced design, use the same size flange plates throughout on all girders to eliminate the problems of getting the wrong flange sizes on a given girder.

Estimate field splice locations at approximately the 7/10 point of continuous spans.

On haunched plate girders the length of parabolic haunch is approximately 1/4 of the span length. The haunch depth is 1 1/2 times the mid span depth.

(3) Determine the Dead Loads, Live Loads, and Live Load Distribution Factors for each member. Add the weight of a possible wearing surface to composite dead load.

(4) Determine the Composite Section for each Member and estimate the Range of Composite Action for each span.

On continuous spans estimate the end of composite action at a distance of approximately 2/10 the span length from the pier. On simple spans use composite action throughout the span.

(5) Input the Preliminary Design into a Computer Program.

(6) The following Steps are taken using the Output from the Computer Program:

Check the loads of the interior and exterior members to see if one or both members are to be designed.

Check the live load deflection of the initial design.

Establish the size of flange plates, web plates, rolled beams and cover plates. Check the bracing of the negative moment flange. The allowable stress may be reduced on the compression flange. Determine if a thinner stiffened plate girder web or a thicker unstiffened web is to be used. On deep plate girders the thinner stiffened web is more economical. The use of a slightly heavier rolled section with an increased section modulus eliminates small cover plate requirements. Wide flange beams are used in preference to I-beams.

Establish butt splice and field splice locations and the length of cover plates. Where a change in steel section is needed, plate girder flanges are butt spliced rather than cover plated.

For a rolled beam consider a shop welded butt splice or adding cover plates to increase section size. Also, check the maximum rolling lengths of plates to see if additional butt splices are required.

Determine the range of composite action for each span. This is the point where shear connectors are omitted or where positive dead load and live load moment do not occur.

The stiffness of the composite section is used for determining live load moments and shears. Dead load moments and shears are based upon the stiffness of the noncomposite steel section.

(7) Input the Proposed Final Design into a Computer Program.

For plate girders include the change in section at butt splices but do not include the change in weight. The fabricator may assume the cost of extending the heavier plate and eliminating the butt splice. The fabricator has used this option on numerous occasions. Shim plates are provided at the bearing to allow for either option.

(8) In Continuous Spans it is necessary to re-input the Size of the Members into the Computer Program when the Final Design is appreciably different than the design originally input.

Changing the stiffness of members in the span or over a support effects the distribution of moments, shears and reactions.

(9) Design the Bearings and Bearing Stiffeners.

The bearings are designed at an early stage in the design since they may govern the width of the bottom flange plates.

(10) Check the Live Load Deflections and Uplift.

(11) Design the Field Splices

(12) Determine the Shear Connector Spacing from Program Print Out.

Refer to Section 24.7(5) for shear connector spacing. Determine the distance from the piers where shear connectors are omitted and field welding to the top flange for construction purposes is not permitted.

(13) Design any Transverse and Longitudinal Stiffeners that are required.

(14) Design the Lateral Bracing and End Diaphragms. Refer to Standards 24.3 through 24.6. Consideration must be given to connection details susceptible to fatigue crack growth.

(15) Determine the Dead Load Deflections, Blocking, Camber, Top of Steel and Top of Slab Elevations with the aid of the Steel Girder Geometrics Computer Program.

(16) Analyze the Structure for Stresses caused during erection and construction for possible overstresses. Check the Lateral Bracing without the Deck Slab.

(17) Determine the Slab Pouring Sequence. Refer to Standard 24.11. Determine the maximum amount of Concrete that can be poured in a day. Determine Deflections based on the Proposed Pouring Sequence.

24.7 COMPOSITE DESIGN AND SHEAR CONNECTORS

(1) Composite Action

Composite design in steel pertains to structures composed of steel beams or girder with concrete slabs connected by shear connectors. The shear connectors prevent slip and vertical separation between the bottom of the slab and the top of the steel member. Unless temporary shoring is used the steel members deflect under the dead load of the wet concrete before the shear connectors become effective. Temporary shoring is not used in Wisconsin, therefore, composite action refers to the live load and portions of dead load placed after the concrete deck has hardened. The composite section in the positive moment area is the concrete in compression. In the negative moment area the composite section is the bar steel reinforcement in the slab. However, the effect of the composite section in the negative moment region is not used and shear connectors are not placed in this region. Composite sections are proportioned to have the neutral axis lie below the top flange of the steel member.

(2) Values of n for Composite Design

The effective composite concrete slab is converted to an equivalent steel area by dividing by n. For f'c = 4 ksi (28 MPa), use n = 8.

f'c = minimum ultimate compressive strength of the concrete slab at 28 days.

n = ratio of modulus of elasticity of steel to that of concrete.

The effect of creep is to be considered in the design of a composite member which has dead loads acting on the composite section. In such structures, stresses and horizontal shears produced by dead loads acting on the composite section are computed for a value of n as given above or for 3n, whichever gives the higher stresses and shears.

(3) Composite Section Properties

The minimum effective slab thickness is equal to the nominal slab thickness minus 1/2" (15 mm) for wearing surface. The maximum effective slab width is defined by AASHTO specifications.

(4) Computation of Stresses

A. Composite Stresses

$$f_b = \frac{DLM(Slab\&\,Steel)}{S(steel)} + \frac{LLM(Traffic)}{S(composite)} + \frac{DLM(SDW, Curbs)}{S(composite)} + \frac{LLM(Pedestrian)}{S(composite)}$$

B. Noncomposite stresses

$$f_b = \frac{DLM(Slab\&\,Steel)}{S(steel)} + \frac{LLM(Traffic)}{S(steel)} + \frac{DLM(SDW, Curbs)}{S(steel)} + \frac{LLM(Pedestrian)}{S(steel)}$$

where: f_b is the computed steel bending stress
DLM is the dead lead moment
LLM is the live load moment
S is the elastic section modulus

(5) Shear Connectors

| Refer to Standard 24.2 for shear connector details. Three shop or field welded 7/8" (22 mm) diameter studs, 4" (100 mm) long are placed on the top flange. The studs are equally spaced with a minimum clearance of 1 1/2" (40 mm) from the edge of the flange. On girders having thicker haunches where stud embedment is less than 2" (50 mm) into the slab, use longer studs to obtain the minimum embedment.

Shear connectors are carried to the point nearest the support where positive moment for combined dead and positive live loading exists. This termination point is used; although theoretically, the shear connectors may be terminated when the steel section can carry the positive moment. The allowable stress in a tension flange is reduced by the fatigue criteria if shear connectors are attached. The top flange is the tension flange for negative moment which may control the flange plate size. Connectors which fall on the flange field splice plates are respaced near the ends of the splice plate. The maximum spacing of shear connectors is 2' (600 mm). Begin connector spacings 9" (230 mm) min. from centerline of abutments.

The shear connector spacing at the tenth points along the span can be determined by using the maximum value of composite moment of inertia in the span. Shear connector spacing is determined by the formula based on fatigue and horizontal shear found in AASHTO.

The number of shear connectors required for fatigue are checked to insure that adequate connectors are provided for ultimate strength. In most cases the connector spacing, three per set, using output based on fatigue requirements is more than adequate for ultimate strength design requirements. Additional shear connectors may be required for ultimate strength design on structures having spans in excess of 150' (45 m).

The number of shear connectors required for ultimate strength is checked between points of maximum positive moment and the adjacent end supports (N_1). The formulas given in the AASHTO specification are applied in the design example for shear connectors.

Additional shear connectors are required at the point of contraflexure when reinforcement steel embedded in the concrete is not used in computing section properties for negative moments. The additional connectors are required to fully develop the slab stresses. The number of additional connectors is computed from the formula in AASHTO.

The additional connectors shall be placed adjacent to and centered on either side of the dead load point of contraflexure within a distance equal to one-third the effective slab width.

24.8 FIELD SPLICES

(1) Computer Program

A computer program is used to design the field splices for wide flange beams and plate girders. Allowable Stress Design is used by the program. The computer outputs the following list of design information.

Net composite and noncomposite section moduli.

Actual stresses, range of stresses, and allowable fatigue range stresses.

Required number of vertical bolt lines for one side of the web splice and the number of bolts per line.

Required net area of top and bottom flange splice plates and the number of bolts for one side of the flange splice.

(2) Location of Field Splice

Field splices shall be placed at the following locations whenever it is practical:

At or near a point of dead load contraflexure.

So that the maximum rolling length of the flange plates is not exceeded, thus eliminating one or more butt splices.

At a point where the fatigue in the net section of the base metal is held to a minimum.

To limit section length to120' (36 m) between splices unless special conditions govern.

(3) Splice Material

The splice material is the same as the members being spliced. Generally, 3/4" (M20) diameter
high strength A325 bolted friction-type connectors are used unless the proportions of the structure warrant larger diameter bolts.

(4) Design

All steel girder field splices on structures that are to be painted shall be designed using an allowable working shear stress for A325 bolts of 15 ksi (103 MPa). For unpainted blast
| cleaned steel or steel with organic zinc paint, field splices shall be designed using a shear stress of 23 ksi (158 MPa).

A. Web Splice

The method used to design the web splice is taken from "Design of Steel Structures" by Gaylord and Gaylord. It is based on the following equation:

$$P = \frac{(R)(n)}{t\sqrt{(s^2 + v^2)}}$$

where:
P is the vertical spacing (pitch) of bolts equally spaced.
R is the allowable force on one bolt in double shear
n is the number of vertical bolt lines on one side of splice
t is the web thickness
s is the maximum bending stress in the web
v is the average shear stress in the web

Two separate designs are output by the program for the web splice. One is referred to as the 75% criteria and the other as the average stress criteria. In the above equation s and v are set equal to the following values:

75% CRITERIA

s = 75% of .55 times the yield stress of the flange. If top and bottom flange have different yield stresses, the higher yield stress is used.
v = 75% of the input "Allowable Shear Stress". *

AVERAGE STRESS CRITERIA

s = The average of the maximum bending stress and .55 times the yield stress of the flange. Since there are four flanges, four "s" values are computed and used to compute four values of "p".
v = The average of the actual shear stress (computed from the input "Total Shear") and the input "Allowable Shear Stress". Two "v" values are computed (one for each side of the splice) and used with the corresponding four "s" values.

* For Plate Girders see AASHTO Figure 1.7.43D1 or Figure 1.7.43D2. For Rolled Beams see AASHTO Table 1.7.1A. Use 12 ksi (80 MPa)

The design based on the average stress criteria is obtained by first selecting the required pitch on each side of the web. The computed pitches on one side of the web at the top and bottom flanges are compared and the smallest is the one that is saved. From the two pitches (one on each side of the web) remaining, the program uses the larger one for the final web splice design. The larger pitch is used because the program designs for the weaker of the two pieces being spliced.

B. Flange Splice

The net area of the flange splice is determined from the following equation:

$$A = \frac{F}{s}$$

where: A is the net area required for the flange splice plates. This is the total net area required for the splice plates on both sides of the flange. (Top and bottom of flange). The area of the splice plates on both sides of the flanges should be approximately the same.

F is the force in the flange obtained by multiplying the "Net Flange Area" by the following four stresses:

1. 75% of .55 times the yield stress of the flange.

2. The average of the maximum bending stress and .55 times the yield stress of the flange.

3. The range of stress in the flange allowed for truck loading. This is used to determine the net area of flange splice plates required to satisfy the allowable fatigue stress range.

4. The range of stress in the flange allowed for lane loading.

s is equal to .55 times the yield stress of the splice material for Number 1 and 2 above, and the allowable fatigue range for Number 3 and 4.

Using the four stresses mentioned above, four values of "A" will be computed for each flange. The maximum of these four values for each flange are then compared to the maximum of the four values of the adjacent flange and the minimum of the two maximums are output for the top flange and bottom flange. These two values are the "REQUIRED TOP NET AREA" and the "REQUIRED BOTTOM NET AREA".

The number of bolts required for the flange splices is calculated from the following equation:

$$N = \frac{F}{(\pi D^2/2)(F_v)}$$

where: N is the number of bolts required. ("N" is rounded up to the next highest even number).

F is the force in flange calculated by multiplying the "Net Flange Area" by the greater of the following two stresses.

1. Seventy five percent of .55 times the yield stress of the flange.

2. The average of the maximum bending stress and .55 times the yield stress of the flange.

D is the bolt diameter.

F is the allowable shear for a friction-type connection with a standard type hole.

The number of bolts required in the top flange splice plates is determined by selecting the minimum required for the two top flanges. The minimum number is selected because the program designs for the weaker of the two pieces being spliced. The number of bolts required in the bottom flange splice plates is determined by the same method used for the top flange splice plates.

24.9 BEARING STIFFENERS

Bearing stiffeners are placed normal to the web of the girder except for skew angles of 15° or less. Here, they may be placed parallel to the skew at abutments and piers to support the end diaphragms or cross framing.

For structures on grades of three percent or more, the end of the girder section at joints is to be cut vertical. This is to eliminate the large extension and clearance problems at the abutments.

1. Plate Girders

   Bearing stiffeners are placed over the end bearings of welded plate girders and also over the intermediate bearings of continuous welded plate girders. The bearing stiffeners extend as near as practical to the outer edges of the flange plate. They consist of 2 or more plates placed on both sides of the web. They are ground to a tight fit at the top flange, welded to the web on both sides with a 1/4" (6 mm) minimum fillet weld, and attached to the bottom flange with full penetration groove welds.

2. Rolled Beams

   Bearing stiffeners are placed over the bearings of rolled beams when the unit shear in the web adjacent to the bearing exceeds 75 percent of the allowable shear stress in the web of the beam. The web of the steel beam carries the total external shear, neglecting the effect of the steel flanges and the concrete slab. The shear is assumed to be uniformly distributed across the gross area of the web.

3. Design

   Bearing stiffeners are designed as columns which transmit the entire end reaction to the bearings. For stiffeners consisting of 2 plates, the column section is assumed to consist of the 2 plates and a centrally located strip of web whose width is less than or equal to 18 times the thickness of the web. For stiffeners consisting of 4 or more plates, the column section is assumed to consist of the plates and a centrally located strip of web whose width is equal to that enclosed by the plates plus a width of not more than 18 times the web plate thickness. Use a spacing of 9" (225 mm) center to center of plates on bearing stiffeners consisting of 4 plates. No part of the web is considered to act in bearing on hybrid girders which have a ratio of minimum yield strength of the web to minimum yield strength of the tension flange less than 0.70. An effective length factor, K, of 0.75 is used.

   Bearing stiffeners may be made of Grade 36 (250) steel unless higher strength or weathering steel is required. The allowable compression in the bearing stiffener is determined from the formula in AASHTO:

$$F_b = 16{,}980 - 0.53(KL/r)^2$$
$$(F_b = 117.1 - 0.00368(KL/r)^2)$$

where:	L is the depth of the web, in. (mm)
	K is the effective length factor
	r is the radius of gyration, in. (mm)

The thickness of bearing stiffener plates is not less than specified by AASHTO.

$$t = \frac{b'}{12}(Fy/33{,}000)^{1/2}$$

$$(t = \frac{b'}{12}(Fy/227.54)^{1/2})$$

where:	t is the thickness of one plate, in. (mm)
	b' is the width of one stiffener plate extended as near practical to the outer edge of the flange, in. (mm)
	Fy is the yield point strength of the bearing stiffener.

The bearing pressure on the edge of the bearing stiffener against the bottom flange of the member is checked. The allowable bearing on milled stiffeners and other parts of steel in contact is 0.80 Fy. In calculating the bearing stiffener assume that the web transfers the entire reaction to the bearing stiffener plates. Deduct 1 1/4" (30 mm) from the plate width for each clipped corner required to clear the flange to web weld.

24.10 TRANSVERSE INTERMEDIATE STIFFENERS

Transverse intermediate stiffeners are plates which are welded to the web and compression flange of the member with a tight fit at the tension flange. Indicate on the plans the flange to which stiffeners are welded. The stiffeners are attached to the web with a continuous 1/4" (6 mm) fillet weld. The dead load moment diagram is used to define the compression flange.

If longitudinal stiffeners are required, the transverse stiffeners are placed on one side of the web of the interior member and the longitudinal stiffener on the opposite side of the web. Place intermediate stiffeners on one side of interior members when longitudinal stiffeners are not required. Transverse stiffeners are placed on the inside web face of exterior members. If longitudinal stiffeners are required, they are placed on the outside web face of exterior members as shown on Standard 24.2.

Transverse stiffeners can be eliminated by increasing the thickness of the web. On plate girders under 48" (1225 mm) in depth, consider thickening the web to eliminate all transverse stiffeners. Within the constant depth portion of haunched plate girders over 48" (1225 mm) deep, consider thickening the web to eliminate the longitudinal stiffener and a portion of the transverse stiffeners within the span.

Current AASHTO Specifications limit bending stress where the girder panel is subject to "tension field action" under the simultaneous action of shear and bending moment. If the full permissible shear stress is allowed, bending stress is limited to approximately 75 percent of the maximum allowable. If the shear stress is limited to 60 percent of the maximum allowable, full bending stress is allowed. This can be accomplished by decreasing the transverse stiffener spacing which in turn increases the allowable shear stress.

The minimum size of transverse stiffeners is 5 x 3/8" (125 x 10 mm). Minimum stiffener width, thickness and moment of inertia is specified by AASHTO.

Transverse stiffeners are placed on the inside face of all exterior girders where the slab overhang exceeds 2' (600 mm) as shown on Standard 24.2. The stiffeners are to prevent web bending caused by construction of the deck slab where triangular overhang brackets are used to support the falsework.

If slab overhang is allowed to exceed the recommended 3'-7" (1100 mm) on exterior girders, the web and stiffeners should be analyzed to resist the additional bending during construction of the deck. Overhang construction brackets may overstress the stiffeners. It may also be necessary to provide longitudinal bracing between stiffeners to prevent localized web deformations which did occur on a structure having 5' (1500 mm) overhangs.

For stiffeners placed on one side of the web only the moment of inertia provided is:

$$I = \frac{th^3}{3}$$

where: I is the moment of inertia of one transverse stiffener about the face of the web plate.
t is the thickness of one transverse stiffener.
h is the width of one transverse stiffener.

When the diaphragms are connected to the transverse intermediate stiffeners, the stiffeners are welded to both the tension and compression flanges. This detail is preferred over bolting details which cost $75-100 extra to fabricate (1996 cost). Flange stresses are usually less than the Category C allowable fatigue stresses produced by this detail which the designer should verify.

24.11 LONGITUDINAL STIFFENERS

Longitudinal stiffeners are plates which are placed on the web at 1/5 of the depth of the web from the compression flange. On the exterior members, the longitudinal stiffeners are placed on the outside face of the web as shown on Standard 24.2. If the longitudinal stiffener is required throughout the length of span on an interior member, the longitudinal stiffener is placed on one side of the web and the transverse stiffeners on the opposite side of the web. Longitudinal stiffeners are normally used in the haunch area of long spans and on a selected basis in the uniform depth section.

Where longitudinal stiffeners are used, place intermediate transverse stiffeners next to the web splice plates at a field splice. The purpose of these stiffeners is to prevent web buckling before the girders are erected and spliced.

AASHTO Specifications give the web thicknesses where longitudinal stiffeners are required. Also, AASHTO specifies a minimum stiffener thickness and a minimum stiffener moment of inertia.

The longitudinal stiffener cut-off point is determined from the formula defined by AASHTO.

If computations indicate that the top or bottom flange carries a compressive stress, and the thickness of the web at that point is less than D/170, then a longitudinal stiffener is required at D/5 from that flange. It is possible to have an overlap of longitudinal stiffeners at D/5 from the top flange and D/5 from the bottom flange due to the variation between maximum positive and maximum negative moment.

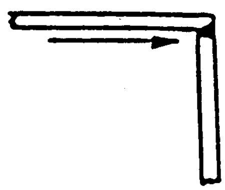
Longitudinal stiffeners are extended full length on exterior girders for aesthetics.

24.12 CONSTRUCTION

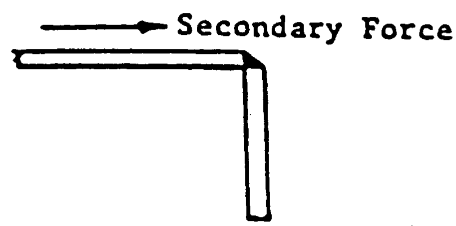
When the deck slab is poured, the exterior girder tends to rotate between the diaphragms. This problem may result if the slab overhang is greater than recommended and/or if the girders are relatively shallow in depth. This rotation causes the rail supporting the finishing machine to deflect downward and changes the roadway grade unless the contractor provides adequate lateral timber bracing.

Stay in place steel forms are not recommended for use. Steel forms have collected water that permeates through the slab and discharges across the top flanges of the girders. As a result the flanges are corroding. Since there are several cracks in the slab, this is a continuous problem.

Where built up box sections are used, full penetration welds provide a stronger joint than fillet welds and give a better looking joint.



Full Penetration Weld



Fillet Weld

The primary force of the member is tension or compression along the axis of the member. The secondary force is a torsion or racing force on the member cross section which produces a shearing force across the weld.

During construction holes may be drilled in the top flanges in the compression zone to facilitate anchorage of posts for the safety lines. The maximum hole size is 3/4" (20 mm) ϕ and prior to pouring the concrete deck, a bolt shall be placed in each hole.

24.13 PAINTING

The final coat of paint on all steel bridges shall be an approved color. Exceptions to this policy may be considered on an individual basis for situations such as scenic river crossings, unique or unusual settings, or local community preference. The District is to submit requests for an exception along with the Structure Survey Report. The colors available for use on steel structures are shown in Chapter 9.

Unpainted weathering steel is used on bridges over streams and railroads. All highway grade separation structures require the use of painted steel as unpainted steel is subject to excessive weathering from salt spray distributed by traffic. On weathering steel bridges, the end 6' (1800 mm) of any steel adjacent to either side of an expansion joint and/or hinge is required to have two shop coats of paint. The second coat is to be brown color similar to rusted steel. Do not paint the exterior girder faces for aesthetic reasons but paint the hanger bar on the side next to the web. If the girder web depth exceeds 6' (1800 mm), paint the girder ends the distance of the web depth. Additional information on Painting is presented in Chapter 9.0 - Materials.

24.14 FLOOR SYSTEMS

In the past floor systems utilizing two main girders were used on long span structures. Current policy is to use multiple plate girder systems for bridges having span lengths up to 400' (120 m). Multiple girder systems are preferred since they are redundant; i.e., failure of any single member will not cause failure of the structure.

In a two girder system the main girders are designed equally to take the dead and live load unless the roadway cross section is unsymmetrical. The dead and live load carried by the intermediate stringers is transferred to the floor beams which transmit the load to the main girders. In designing the main girders it is an acceptable practice to assume the same load distribution along the stringers as along the girder and ignore the concentrated loads at the floor beam connections.

The design criteria used for the girders with the floor systems is the same as the criteria used for plate girders and rolled sections. Pay particular attention to the connection details to be sure they are adequate and to the lateral bracing requirements and connections.

24.15 BOX GIRDERS

Initial considerations for the design of box girders indicated a material savings in the structure due to a more liberal live load distribution factor and to lesser requirements for transverse bracing between girders. Box girders present a smooth, uncluttered appearance under the bridge deck due to the lack of transverse bracing and to their closed section.

With these apparent benefits Wisconsin designed some trapezoidal box girders to see what the steel fabricators and bridge contractors experience would indicate.

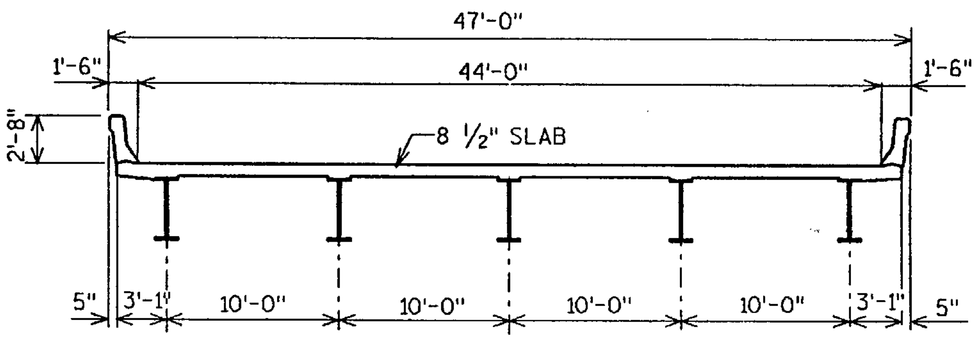
These structures are now open to traffic in Wisconsin. Due to the geometric limitation of plate sizes to control buckling, there was no material savings with box girders. It also appears that more intermediate lateral bracing was required between the girders to transfer wind loads than was provided.

In the design of box girders, the concrete slab was designed as a portion of the top flange and also as the support between the two girder webs. During construction the box girder required additional bracing to support the web until the slab was in place. This was either temporary bracing removed after the slab was cured or permanent bracing left in place. In either case, it was an additional cost item. In addition, temporary bracing was required between the girders to transfer wind loads until the deck slab was poured.

Current experience shows that box girders may require more material than conventional plate girders. On longer spans additional bracing between girders is required to transfer lateral loads. Therefore, the use of box girder bridges is not currently recommended.

24.16 DESIGN EXAMPLE

(This example uses English units).



CROSS SECTION

HS20 live loading, 2 span continuous 180'-0, 180'-0

(10'-0") - 1/2 (14" Top Flg) = 9'-5" eff. span.

Use 8 1/2" slab.

Make all girders same.

Full curb load (parapets) to exterior girders.

Structure is designed for a future wearing surface of 20 pounds per square foot.

Allowable Design Stresses:

Concrete Masonry:	Slab .... f'_c	=	4,000 psi
	All other .... f'_c	=	3,500 psi
Bar Steel Reinforcement, Grade 400 (60)	f_y	=	60,000 psi
Structural Carbon Steel Grade 250 (36)			
Minimum Yield Strength	F_y	=	36,000 psi
Structural Low-Alloy Steel Grade 345(50)			
Minimum Yield Strength	F_y	=	50,000 psi

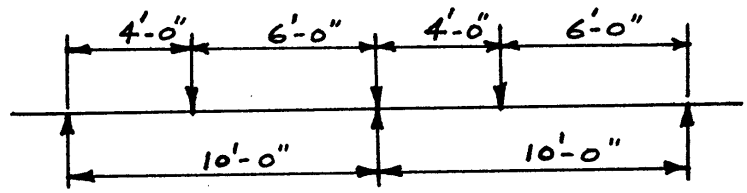
COMPUTER INPUT

Interior Girders:

LL Distr. Factor = $\frac{10.00}{5.5}$ = 1.82 wheels/girder

LLR = $\frac{(4.0 + 10.0 + 6.0)}{10.0}$ = 2.0 wheels/girder

Typical interior girder spacing with wheel loadings for Live Load Reaction



Concrete Dead Load:

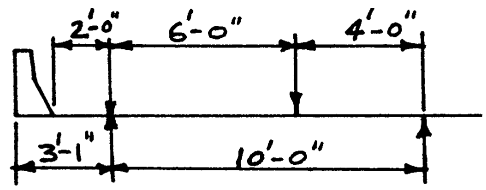
Slab	= 8.5"/12 x 10.0' x 150	=	1063
Diaphragms & Miscellaneous		=	50
Total Load		=	1113 #/Ft.

Curb Dead Load = (10.0)(20) = 200 #/Ft. due to possible placement of future wearing surface. Curb dead load is to be placed on the composite section.

Exterior Girders

LL Distr. Factor = $\frac{10.0}{4 + (0.25)(10.0)}$ = 1.54 wheels/girder

Location of wheel loadings for Live Load Reaction



LLR = $\frac{(4.0 + (10.0)}{(10.0)}$ = 1.4 wheels/girder

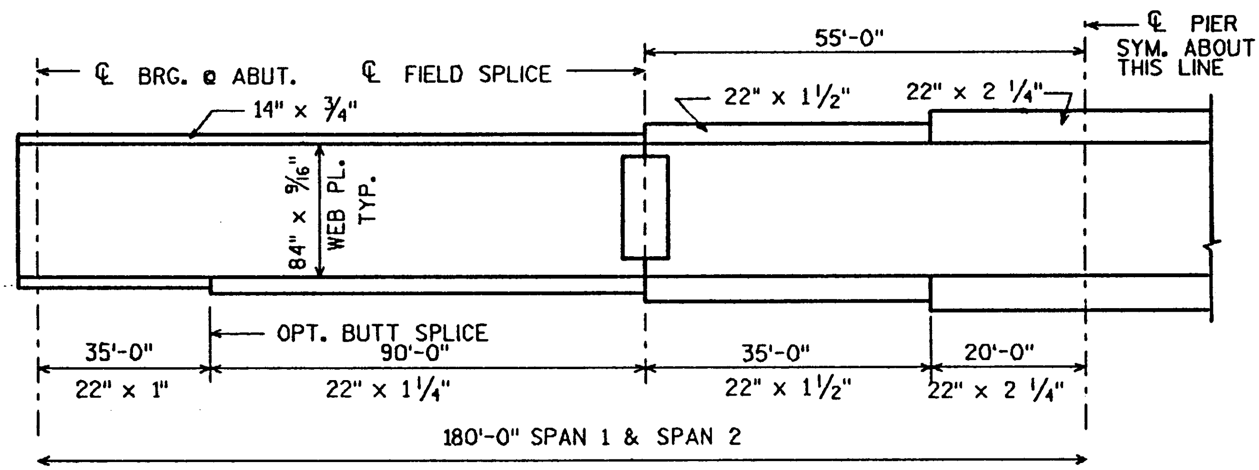
Concrete Dead Load:

Slab	=	8.5"/12 x 8.08' x 150	=	859
Diaphragms & Misc.	=			40
Total Load	=	899#/Ft	=	900#/Ft

Curb Dead Load = 338#/Ft + (7.0')(20) = 478#/Ft.

COMPUTER OUTPUT

Final Plate Sizes



Live Load Deflections

From the SIMON computer output for Interior and Exterior girders, Span 1, the live load deflections are tabulated below:

Actual maximum deflection for interior girders = 1.32"

Actual maximum deflection for exterior girders = 1.35"

Checking with the allowable deflection of L/1250 = $\frac{180' \times 12}{1250}$ = 1.728"

The actual deflection is well within allowable limit.

Bearing Stiffener Design - At Pier

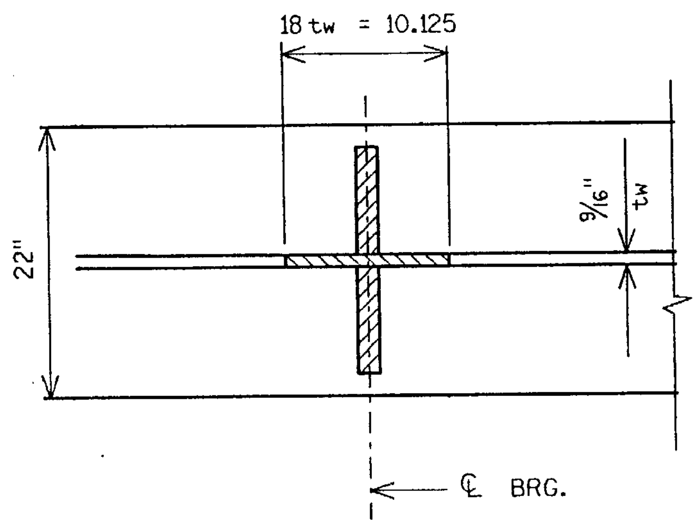
Interior Girder Reaction

(Dead Load + curb load + live load brg. lane) reaction = 567.9 kips

Interior girder reaction controls

Cross section of girder at pier

Top and bottom flanges	22" x 2 1/4"	(Grade 50)
Web Plate	84" x 9/16"	(Grade 50)



Referring to above sketch, try 2 Plates - 10 1/4" x 1 3/8" (Grade 36)

$$t_{min} \geq \frac{10.25}{12} (36/33)^{1/2} = 0.892" < 1\ 3/8" \text{ o.k.}$$

Stiffener column area is as follows:

$$A = (2)(10.25 \times 1.375) + (10.125 \times .5625) = 33.88 \text{ sq. in.}$$

$$f_{(actual)} = \frac{567.9 \times 1000}{33.88} = 16{,}762 \text{ psi}$$

$$f_{(allow)} = 23{,}580 - 1.03\,(KL/r)^2 \text{ where}$$

$K = 0.75$
$L = 84"$

$r = (I/A)^{1/2}$

$I = bh^3/12 = (1.375)(21.0625)^3/12 = 1{,}070.6 \text{ in.}^4$

$A = 33.88$ Sq. In.

$r = (1070.6/33.88)^{1/2} = 5.62$ in.

$f_{(allow)} = 23{,}580 - 1.03\,[(0.75)(84/5.62)]^2$

$= 23{,}450 \text{ psi} > 16{,}762\ f_{(actual)}$ o.k.

Stiffener bearing area deducting for 1 1/4" clipped corners is as follows:

$A = (2)(10.25 - 1.25)(1.375) = 24.75 \text{ in.}^2$

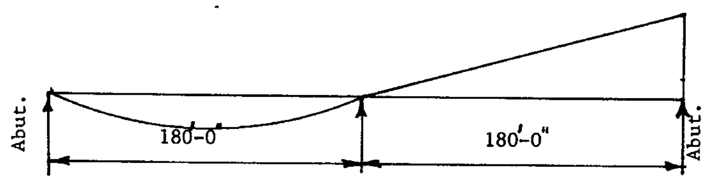
$$f_{(brg)}\ \frac{567.9 \times 1000}{24.75} = 22{,}945 \text{ psi}$$

$f_{(allow)} = 0.8\,Fy = 28{,}800 \text{ psi} > f_{(brg)}$ o.k.

<u>Use: 2 plates 10 1/4" x 1 3/8" (Grade 36)</u>

Check Uplift

HS-20 Live Load



Check uplift at east abutment, by employing influence line coefficients for shear (from continuous beam analysis program).

Lane Loading = (103.9 x 0.1 x .640) + (88 x .001 x 26) = 8.94 kips ↑

Truck Loading = (88 x .001 x 32) + (84.7 x .001 x 32) + (79 x .001 x 8)
= 6.16 kips ↑

Lane loading governs the uplift at 8.94 kips ↑

$$\text{Impact} = \frac{50}{180.0 + 125} \times (100) = 16.4\%$$

The total uplift force is as follows:

= (loading)(impact)(no. of lanes)(lane reduction)(100% increase)
= (8.94)(1.164) x (3) x (0.9) x 2 = 56.19 kips-

The dead load reaction is as follows:

= Exterior girders + Interior girders

= (2)(74.7) = 3 (88.1) = 413.7 kips ↓

Uplift does not occur at the abutments, permanent hold down devices are not required.

Field Splice Design

Top flg.	14 x 3/4 (Grade 50) spliced to 22 x 1 1/2 (Grade 50)
Web	84 x 9/16 (Grade 50) spliced to 84 x 9/16 (Grade 50)
Bottom Flg.	22 x 1 1/4 (Grade 50) spliced to 22 x 1 1/2 (Grade 50)

All field splice plates are Grade 50 steel and A325 - 3/4 inch diameter bolts are used throughout an allowable double shear value of 16 kips per square inch.

Top Flange Splice

From Computer output: Net Area required = 7.88 in^2 and 16 bolts/side (2 bolts/line)

(14)(3/8) - (2)(3/4 + 1/16)(3/8) = 4.64 in^2
(2)(6)(3/8) - (2)(3/4 + 1/16)(3/8) = 3.89 in^2

Total Area = 8.53 $in.^2$ > 7.88 in^2

((1 1/4) + (7)(2 1/2) + (1 1/2)) x 2 = 3'-4 1/2" Long

Use: 1 Plate 14 x 3/8 x 3' - 4 1/2" Long
2 Plates 6 x 3/8 x 3' - 4 1/2" Long

32 - 3/4" H.S. Bolts

Bottom Flange Splice

From Computer output: Net area required = 20.44 $in.^2$ and 40 bolts/side (4 bolts/line)

(22)(3/4) - (4)(3/4 + 1/16)(3/4) = 14.06 in^2
(2)(10)(1/2) - (4)(3/4 + 1/16)(1/2) = 8.37 in^2

Total Area Provided = 22.43 in^2 > 20.44 in^2

((1 1/4) + (9)(2 1/2) + (1 1/2)) x 2 = 4'-2 1/2 Long

Use: 1 Plate 22 x 3/4 x 4' - 2 1/2" Long
2 Plates 10 x 1/2 x 4' - 2 1/2" Long

80 - 3/4" H.S. Bolts

Web Splice

Computer output: 2 vert. lines per side with 29 bolts each.

((1 1/4) + (1)(2 1/2) + 1 1/2) 2 = 10 1/2" wide

Use: 2 Plates = 10 1/2" x 3/8 x 5'-2 Long
4 vert. lines, 29 bolts per line pitched at 2 3/4"
116 - 3/4" H.S. Bolts

Transverse Intermediate Stiffener Design

Transverse intermediate stiffener spacing is limited to one half the web depth at the abutments and the maximum spacing is limited to 1.5 times the web depth. Current AASHTO Specifications require transverse stiffeners if the web thickness is less than D/150.

Reference is made to Section 24.10 for discussion of current computer design. SIMON computer output print distances in feet from left end of span to each transverse stiffeners as follows:

3.5 14.0 117.0 127.5 138.0 148.5 159.0 169.5

Stiffener Size Criteria

AASHTO Article 10.48.5.5 has the requirement of the criteria for width-to-thickness ratio and moment of inertia of transverse stiffeners.

SIMON Computer Program prints out size of transverse stiffener plates. This sample problem requires 6 1/4" x 1 1/8" plates.

The required moment of inertia according to AASHTO requirement Article 10.34.4.7 and 10.48.5.5 is:

$I\ (req'd) = (d_o)(t_w)^3(J)$ where:

I	=	Minimum permissible moment of inertia of any type of transverse intermediate stiffeners in inches^4.
J	=	Required ratio of rigidity of one transverse stiffener to that of the web plate.
tw	=	Thickness of the web plate in inches.
D	=	Unsupported depth of web plate between flange components in inches.
do	=	Actual distance between stiffeners in inches.

J = $2.5\ (D/do)^2\ - 2.0$ ³ 0.5

$2.5\ (84/126)^2\ - 2.0 < 0.5$

Use 0.5

$I\ (\text{req'd}) = 126 \times (.5625)^3 \times .5 = 11.22\ in^4$

$I\ (\text{prov'd}) = \dfrac{(t)(h)^3}{3} = \dfrac{1.125(6.25)^3}{3} = 91.5\ in^4$ o.k.

Use: 6 1/4 x 1 1/8 stiffeners at both spans

Longitudinal Stiffener Design

Past experience shows that the steel girder bridges with longitudinal stiffeners are a constant source for fatigue crack problems. Current policy is to avoid longitudinal stiffeners at all steel girders. However, if longitudinal stiffeners are absolutely unavoidable in steel girder design, AASHTO Article Nos. 10.48.6.1, 10.48.6.2 and 10.48.6.3 should be followed.

Slab on Steel Girder Design

Reference is made to Design Manual Figure 17.1 and Table 17.2 for slab reinforcement location and bar steel reinforcing requirements. The effective span and slab thickness are required to use Table 17.2.

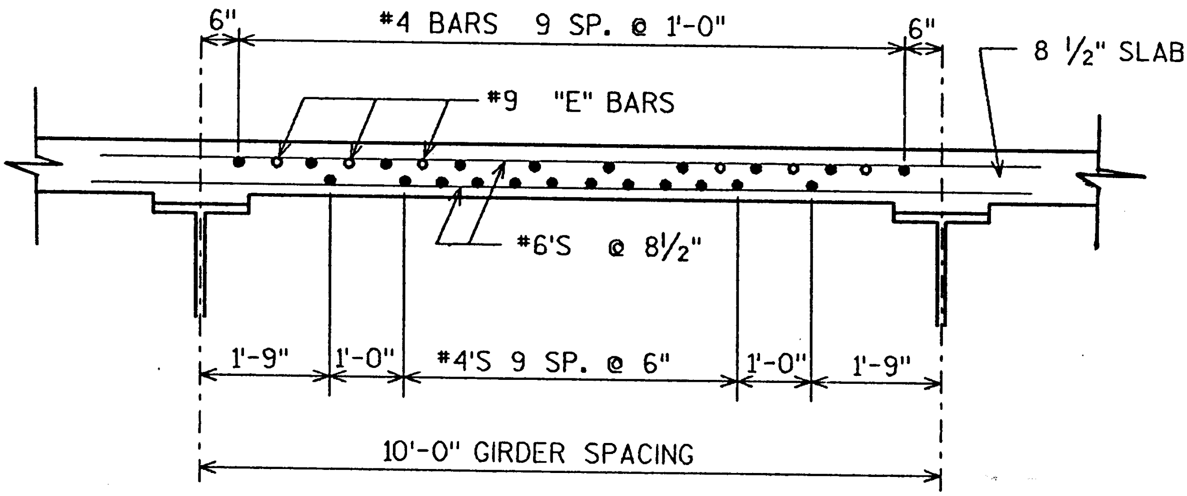
Effective Span = 10'-0" - (1/2)(14") = 9'-5"
Slab Thickness (T) = 8 1/2 inches

From Table 17.2, the following reinforcement is required:

"A" and "B" Bars: #6's @ 8 1/2 Center to Center
"C" Bars: #4's (12 Total)
"D" Bars: #4's (10 Total)
"E" Bars: #9's (6 Total)

Note in addition to "D" bars, "E" bars are required in the negative moment regions of continuous spans. The minimum reinforcement including the longitudinal distribution reinforcement must equal or exceed 1 percent of the cross-section area. Two-thirds of this required reinforcement is to be placed in the top layer of the slab within the effective width.

The area required ("E" Bars) in the negative moment regions is supplied by using 6 - #9's placed within effective width of girders. These "E" Bars are extended from the negative moment regions of continuous spans at least 40 bar diameters beyond the additional shear connectors into the positive moment regions.



Shear Connector Design

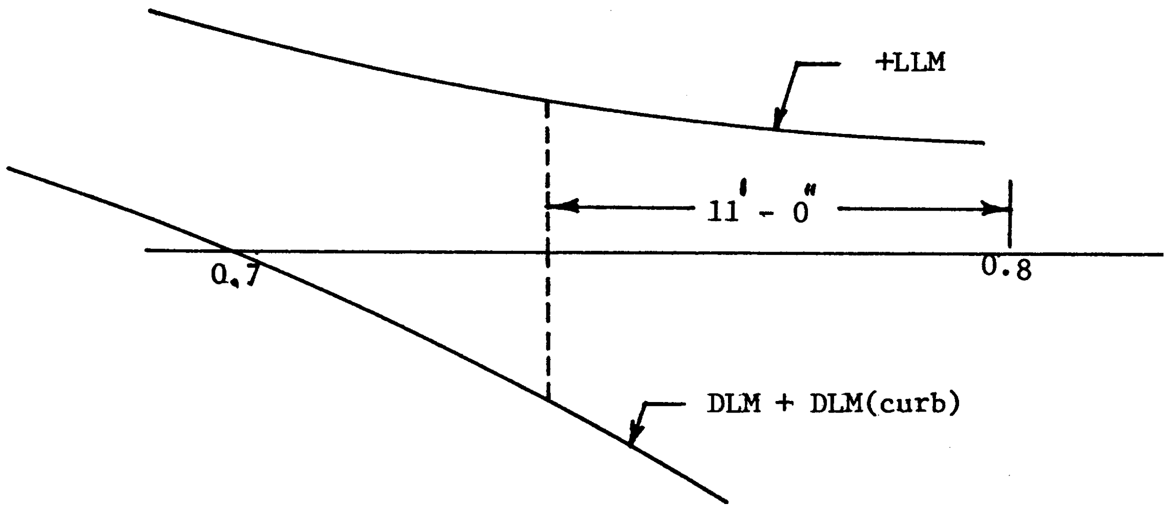
Shear connectors are welded in sets of three to the top flange in either the shop or field. The shear capacity of (3)-4" x 7/8" f connectors is 23.8 kips. The maximum shear connector spacing by AASHTO Specifications, 10.38.5.1, is 2 feet and they are terminated nearest to the continuous support where positive moment for combined dead load and positive live loading exist.

The following shear connector spacings are given on the SIMON Computer output for Span 1:

6 Spaces @ 1'-9"

80 Spaces @ 2'-0"

The shear connector termination point in Span 1 is located between the 7th and 8th tenth points as follows:



By scale, the ordinates are equal to the 0.74 point of Span 1 which is approximately 47 feet from the pier. For detailing purposes, the connectors are terminated at an even spacing beyond the 0.74 point.

The following formulas from AASHTO Article 10.38.5.1.2 are used to determine the shear connector requirements for ultimate strength. The number of connectors (N_1) required from the points of maximum positive moment and adjacent end supports are as follows:

$N_1 = \dfrac{P}{\phi Su}$ where P is the smaller

of P_1 or P_2, ϕ = 0.85, and Su is the ultimate strength capacity of the shear connector.

$P_1 = A_s Fy, \quad A_s = 85.25 \text{ in}^2$

$P_1 = (85.25)(50{,}000) = 4{,}262{,}500$ #

$P_2 = 0.85 f'_c b\, c$

$= (0.85)(4000)(96)(8.0) = 2{,}611{,}200$ #

$P = P_2 = 2{,}611{,}200$ #

$Su = 0.4 (d)^2 (f'c\, Ec)^{1/2}$ for studs

$= (0.4)(7/8)^2 \quad ((4000)(3{,}600 \times 1000))^{1/2} = 36{,}750\ \#$

$N_1 = \dfrac{2{,}611{,}200}{(0.85)(36{,}750)} = 83.6$ studs required

The actual number based on fatigue and horizontal shear from the 0.0 to 0.4 point of Span 1 or 2 is:

6 Spaces @ 1'-9"	=	10'-6"
30 Spaces @ 2'-0"	=	60'-0"
1'-0" from end	=	1'-0"
		71'-6"

Total Number of Studs provided = (37)(3 per set) = 111 studs o.k.

Additional shear connectors to develop slab stress at the point of contra-flexure are required by AASHTO Specifications, Article 10.38.5.1.3, when reinforcement steel embedded in the concrete is not used in computing section properties for negative moments. The additional sets are computed from the following formula:

$N_c = A^s_r \times (fr/Zr)$

$A^s_r = 10.40\ in^2$

$N_c = (10.40)(10{,}000) / 24{,}346 = 4.3$ sets

Use six extra sets of three studs; place three sets on either side of the field splice.

<u>Lateral Bracing Check</u>

Determine the need for Lateral Diagonal Bracings (Lower lateral bracings).

Depth of Section = 2.67' + 0.71 + 7.00 = 10.4 ft^2/ft

Wind Force = 50#/ft^2 x 10.4 ft^2/ft = 520#/ft. Apply 1/2 of this (260 #/ft) to lower Lateral System. For the sake of simplicity in this computation assume all 10th points of span considered:

f_{DL} = 18 ksi f_{LL} = 9 ksi (Assumed maximum values) from the support. Try for a spacing of 25' using AASHTO Specification 10.20.2.

$R = (.2272xL\text{-}11)\ S_d^{\ 2/3}$

$R = (.2272 \times 180 - 11)\ 25^{\ 2/3} = 29.9/8.5 = 3.5$

$F_{cb} = 72M_{cb}/t_f b_f^{\,2}$

$M_{cb} = .08xWxS_d^{\,2} = .08 \times .260 \times 25^2 = 13.0$ kips

$F_{cb} = 72x13.0/1.25x(22)^2 = 1.55$ ksi

$F = R\,F_{cb} = 3.5x1.55 = 5.43$ ksi

Group II $D + W = 18 + 5.43 = 23.43$ ksi

Group III $D + L + .3W = 18 + 9 + .3 \times 5.43 = 28.63$ ksi

f_b (Allowable) $= 27 \times 1.25 = 33.75$ ksi $>$ 28.63 ksi

Lower lateral bracing is not required in the final design condition. Temporary construction loadings may warrant bracing in top flanges for at least during placement of the superstructure. A computation is made below:

$$M_{cb} = .08 \times WxS_d^{\,2} = 0.08 \times \frac{.260}{10.4} \times 7.0x(25)^2 = 8.75 \text{ kips}$$

$F_{cb} = 72M_{cb}/tb^2 = 72x8.75/.75x14^2 = 4.3$ ksi

$F = RF_{cb} = 3.5 \times 4.3 = 15.0$ ksi

F_{DL} Due to Steel Alone = 3.5 ksi

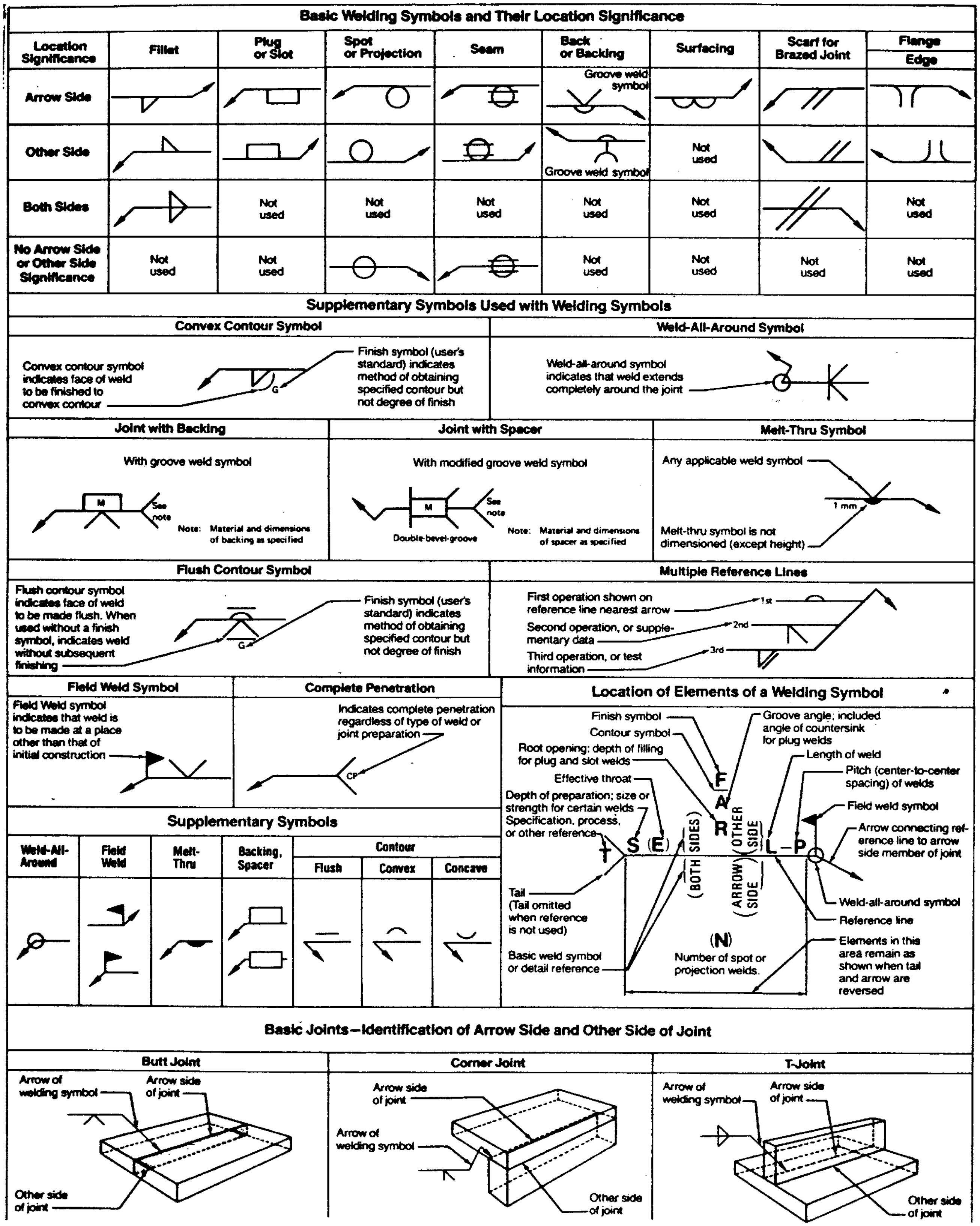
Group II Temporary = 3.5 ksi + 15.0 ksi = 18.5 ksi

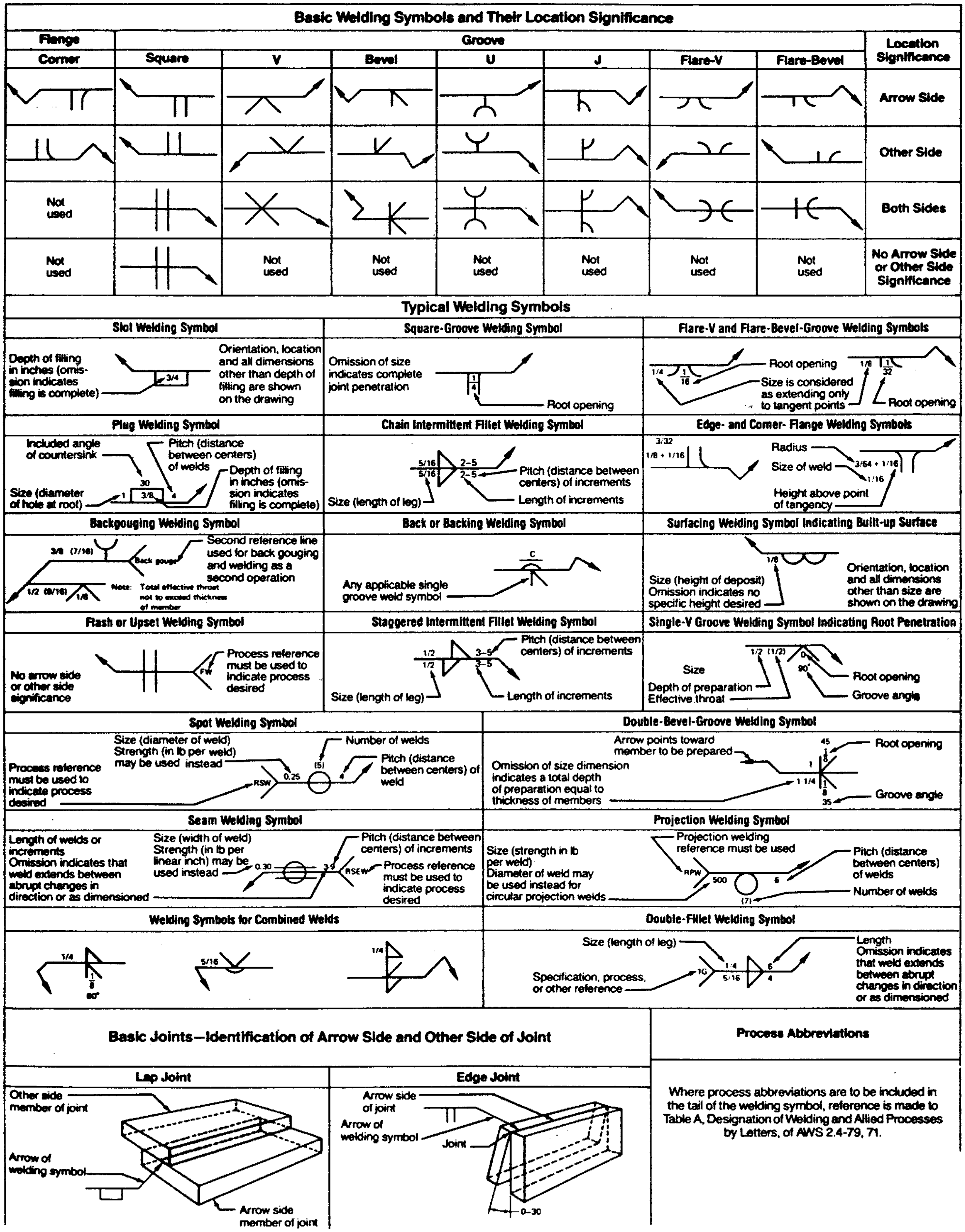
$$\text{Comp. Flg. Flexure } F_b = 27{,}000 - 14.4\ \frac{(25x12)^2}{14} = 20{,}300 \text{ psi}$$

Allowable $F_T = 1.25 \times 20{,}300 = 25{,}375$ psi $>$ 18,500 psi

No temporary bracing is required in this case.

Current practice on this matter is discussed more fully in the Manual Section on Lower Lateral Bracing.


Basic Welding Symbols and Their Location Significance
Location Significance
Fillet
Plug or Slot
Spot or Projection
Seam
Back or Backing
Surfacing
Scarf for Brazed Joint
Flange
Edge
Arrow Side
Groove weld symbol
Other Side
Groove weld symbol
Not used
Both Sides
Not used
No Arrow Side or Other Side Significance
Supplementary Symbols Used with Welding Symbols
Convex Contour Symbol
Convex contour symbol indicates face of weld to be finished to convex contour
Finish symbol (user's standard) indicates method of obtaining specified contour but not degree of finish
G
Weld-All-Around Symbol
Weld-all-around symbol indicates that weld extends completely around the joint
Joint with Backing
With groove weld symbol
M
See note
Note: Material and dimensions of backing as specified
Joint with Spacer
With modified groove weld symbol
Double-bevel-groove
Note: Material and dimensions of spacer as specified
Melt-Thru Symbol
Any applicable weld symbol
1 mm
Melt-thru symbol is not dimensioned (except height)
Flush Contour Symbol
Flush contour symbol indicates face of weld to be made flush. When used without a finish symbol, indicates weld without subsequent finishing
Finish symbol (user's standard) indicates method of obtaining specified contour but not degree of finish
Multiple Reference Lines
First operation shown on reference line nearest arrow
1st
Second operation, or supplementary data
2nd
Third operation, or test information
3rd
Field Weld Symbol
Field Weld symbol indicates that weld is to be made at a place other than that of initial construction
Complete Penetration
Indicates complete penetration regardless of type of weld or joint preparation
CP
Supplementary Symbols
Weld-All-Around
Field Weld
Melt-Thru
Backing, Spacer
Contour
Flush
Convex
Concave
Location of Elements of a Welding Symbol
Finish symbol
Contour symbol
Root opening; depth of filling for plug and slot welds
Effective throat
Depth of preparation; size or strength for certain welds
Specification, process, or other reference
Tail (Tail omitted when reference is not used)
Basic weld symbol or detail reference
Groove angle; included angle of countersink for plug welds
Length of weld
Pitch (center-to-center spacing) of welds
Field weld symbol
Arrow connecting reference line to arrow side member of joint
Weld-all-around symbol
Reference line
Elements in this area remain as shown when tail and arrow are reversed
F
A
R
S (E)
L – P
(BOTH SIDES)
(OTHER SIDE)
(ARROW SIDE)
(N)
Number of spot or projection welds.
Basic Joints—Identification of Arrow Side and Other Side of Joint
Butt Joint
Arrow of welding symbol
Arrow side of joint
Other side of joint
Corner Joint
Arrow side of joint
Arrow of welding symbol
Other side of joint
T-Joint
Arrow of welding symbol
Arrow side of joint
Other side of joint


Basic Welding Symbols and Their Location Significance
Flange
Corner
Groove
Square
V
Bevel
U
J
Flare-V
Flare-Bevel
Location Significance
Arrow Side
Other Side
Both Sides
No Arrow Side or Other Side Significance
Not used
Typical Welding Symbols
Slot Welding Symbol
Depth of filling in inches (omission indicates filling is complete)
Orientation, location and all dimensions other than depth of filling are shown on the drawing
Square-Groove Welding Symbol
Omission of size indicates complete joint penetration
Root opening
Flare-V and Flare-Bevel-Groove Welding Symbols
Root opening
Size is considered as extending only to tangent points
Plug Welding Symbol
Included angle of countersink
Pitch (distance between centers) of welds
Depth of filling in inches (omission indicates filling is complete)
Size (diameter of hole at root)
Chain Intermittent Fillet Welding Symbol
Pitch (distance between centers) of increments
Length of increments
Size (length of leg)
Edge- and Corner- Flange Welding Symbols
Radius
Size of weld
Height above point of tangency
Backgouging Welding Symbol
Second reference line used for back gouging and welding as a second operation
Back gouge
Note: Total effective throat not to exceed thickness of member
Back or Backing Welding Symbol
Any applicable single groove weld symbol
Surfacing Welding Symbol Indicating Built-up Surface
Size (height of deposit) Omission indicates no specific height desired
Orientation, location and all dimensions other than size are shown on the drawing
Flash or Upset Welding Symbol
No arrow side or other side significance
Process reference must be used to indicate process desired
Staggered Intermittent Fillet Welding Symbol
Pitch (distance between centers) of increments
Size (length of leg)
Length of increments
Single-V Groove Welding Symbol Indicating Root Penetration
Size
Depth of preparation
Effective throat
Root opening
Groove angle
Spot Welding Symbol
Size (diameter of weld)
Strength (in lb per weld) may be used instead
Number of welds
Pitch (distance between centers) of weld
Process reference must be used to indicate process desired
Double-Bevel-Groove Welding Symbol
Arrow points toward member to be prepared
Root opening
Omission of size dimension indicates a total depth of preparation equal to thickness of members
Groove angle
Seam Welding Symbol
Length of welds or increments
Omission indicates that weld extends between abrupt changes in direction or as dimensioned
Size (width of weld)
Strength (in lb per linear inch) may be used instead
Pitch (distance between centers) of increments
Process reference must be used to indicate process desired
Projection Welding Symbol
Projection welding reference must be used
Size (strength in lb per weld)
Diameter of weld may be used instead for circular projection welds
Pitch (distance between centers) of welds
Number of welds
Welding Symbols for Combined Welds
Double-Fillet Welding Symbol
Size (length of leg)
Specification, process, or other reference
Length
Omission indicates that weld extends between abrupt changes in direction or as dimensioned
Basic Joints—Identification of Arrow Side and Other Side of Joint
Lap Joint
Other side member of joint
Arrow of welding symbol
Arrow side member of joint
Edge Joint
Arrow side of joint
Arrow of welding symbol
Joint
Process Abbreviations
Where process abbreviations are to be included in the tail of the welding symbol, reference is made to Table A, Designation of Welding and Allied Processes by Letters, of AWS 2.4-79, 71.

Width, mm	Thickness, mm																			
	10	11	12	14	16	18	20	22	25	28	30	32	35	38	40	45	50	55	60	70
300	23.5	25.9	28.3	33.0	37.7	42.4	47.1	51.8	58.9	65.9	70.6	75.4	82.4	89.5	94.2	106.0	117.7	129.5	141.3	164.8
325	25.5	28.1	30.6	35.7	40.8	45.9	51.0	56.1	63.8	71.4	76.5	81.6	89.3	96.9	102.0	114.8	127.5	140.3	153.1	178.6
350	27.5	30.2	33.0	38.5	44.0	49.4	54.9	60.4	68.7	76.9	82.4	87.9	96.2	104.4	109.9	123.6	137.4	151.1	164.8	192.3
375	29.4	32.4	35.3	41.2	47.1	53.0	58.9	64.8	73.6	82.4	88.3	94.2	103.0	111.8	117.7	132.5	147.2	161.9	176.6	206.0
400	31.4	34.5	37.7	44.0	50.2	56.5	62.8	69.1	78.5	87.9	94.2	100.5	109.9	119.3	125.6	141.3	157.0	172.7	188.4	219.8
425	33.4	36.7	40.0	46.7	53.4	60.0	66.7	73.4	83.4	93.4	100.1	106.7	116.8	126.8	133.4	150.1	166.8	183.5	200.1	233.5
450	35.3	38.9	42.4	49.4	56.5	63.6	70.6	77.7	88.3	98.9	106.0	113.0	123.6	134.2	141.3	158.9	176.6	194.3	211.9	247.2
475	37.3	41.0	44.7	52.2	59.7	67.1	74.6	82.0	93.2	104.4	111.8	119.3	130.5	141.7	149.1	167.8	186.4	205.1	223.7	261.0
500	39.2	43.2	47.1	54.9	62.8	70.6	78.5	86.3	98.1	109.9	117.7	125.6	137.4	149.1	157.0	176.6	196.2	215.8	235.5	274.7
525	41.2	45.3	49.4	57.7	65.9	74.2	82.4	90.7	103.0	115.4	123.6	131.9	144.2	156.6	164.8	185.4	206.0	226.6	247.2	288.5
550	43.2	47.5	51.8	60.4	69.1	77.7	86.3	95.0	107.9	120.9	129.5	138.1	151.1	164.0	172.7	194.3	215.8	237.4	259.0	302.2
575	45.1	49.6	54.2	63.2	72.2	81.2	90.3	99.3	112.8	126.4	135.4	144.4	158.0	171.5	180.5	203.1	225.7	248.2	270.8	315.9
600	47.1	51.8	56.5	65.9	75.4	84.8	94.2	103.6	117.7	131.9	141.3	150.7	164.8	179.0	188.4	211.9	235.5	259.0	282.6	329.7
625	49.1	54.0	58.9	68.7	78.5	88.3	98.1	107.9	122.6	137.4	147.2	157.0	171.7	186.4	196.2	220.8	245.3	269.8	294.3	343.4
650	51.0	56.1	61.2	71.4	81.6	91.8	102.0	112.2	127.5	142.9	153.1	163.3	178.6	193.9	204.1	229.6	255.1	280.6	306.1	357.1
675	53.0	58.3	63.6	74.2	84.8	95.4	106.0	116.6	132.5	148.3	158.9	169.5	185.4	201.3	211.9	238.4	264.9	291.4	317.9	370.9
700	54.9	60.4	65.9	76.9	87.9	98.9	109.9	120.9	137.4	153.8	164.8	175.8	192.3	208.8	219.8	247.2	274.7	302.2	329.7	384.6
725	56.9	62.6	68.3	79.7	91.0	102.4	113.8	125.2	142.3	159.3	170.7	182.1	199.2	216.2	227.6	256.1	284.5	313.0	341.4	398.3
750	58.9	64.8	70.6	82.4	94.2	106.0	117.7	129.5	147.2	164.8	176.6	188.4	206.0	223.7	235.5	264.9	294.3	323.8	353.2	412.1
775	60.8	66.9	73.0	85.2	97.3	109.5	121.7	133.8	152.1	170.3	182.5	194.7	212.9	231.2	243.3	273.7	304.1	334.6	365.0	425.8
800	62.8	69.1	75.4	87.9	100.5	113.0	125.6	138.1	157.0	175.8	188.4	200.9	219.8	238.6	251.2	282.6	314.0	345.4	376.8	439.5
825	64.8	71.2	77.7	90.7	103.6	116.6	129.5	142.5	161.9	181.3	194.3	207.2	226.6	246.1	259.0	291.4	323.8	356.1	388.5	453.3
850	66.7	73.4	80.1	93.4	106.7	120.1	133.4	146.8	166.8	186.8	200.1	213.5	233.5	253.5	266.9	300.2	333.6	366.9	400.3	467.0
875	68.7	75.5	82.4	96.2	109.9	123.6	137.4	151.1	171.7	192.3	206.0	219.8	240.4	261.0	274.7	309.1	343.4	377.7	412.1	480.8
900	70.6	77.7	84.8	98.9	113.0	127.2	141.3	155.4	176.6	197.8	211.9	226.1	247.2	268.4	282.6	317.9	353.2	388.5	423.8	494.5
925	72.6	79.9	87.1	101.6	116.2	130.7	145.2	159.7	181.5	203.3	217.8	232.3	254.1	275.9	290.4	326.7	363.0	399.3	435.6	508.2
950	74.6	82.0	89.5	104.4	119.3	134.2	149.1	164.0	186.4	208.8	223.7	238.6	261.0	283.3	298.3	335.5	372.8	410.1	447.4	522.0
975	76.5	84.2	91.8	107.1	122.4	137.7	153.1	168.4	191.3	214.3	229.6	244.9	267.8	290.8	306.1	344.4	382.6	420.9	459.2	535.7
1000	78.5	86.3	94.2	109.9	125.6	141.3	157.0	172.7	196.2	219.8	235.5	251.2	274.7	298.3	314.0	353.2	392.5	431.7	470.9	549.4
1025	80.5	88.5	96.5	112.6	128.7	144.8	160.9	177.0	201.1	225.3	241.4	257.4	281.6	305.7	321.8	362.0	402.3	442.5	482.7	563.2
1050	82.4	90.7	98.9	115.4	131.9	148.3	164.8	181.3	206.0	230.8	247.2	263.7	288.5	313.2	329.7	370.9	412.1	453.3	494.5	576.9

FLAT ROLLED STEEL, Weight per Linear Meter, Kilograms

Width, mm							Thickness, mm												
	10	11	12	14	16	18	20	22	25	28	30	32	35	38	40	45	50	55	60	70
1075	84.4	92.8	101.3	118.1	135.0	151.9	168.8	185.6	210.9	236.3	253.1	270.0	295.3	320.6	337.5	379.7	421.9	464.1	506.3	590.6
1100	86.3	95.0	103.6	120.9	138.1	155.4	172.7	189.9	215.8	241.7	259.0	276.3	302.2	328.1	345.4	388.5	431.7	474.9	518.0	604.4
1125	88.3	97.1	106.0	123.6	141.3	158.9	176.6	194.3	220.8	247.2	264.9	282.6	309.1	335.5	353.2	397.4	441.5	485.7	529.8	618.1
1150	90.3	99.3	108.3	126.4	144.4	162.5	180.5	198.6	225.7	252.7	270.8	288.8	315.9	343.0	361.1	406.2	451.3	496.4	541.6	631.8
1175	92.2	101.4	110.7	129.1	147.6	166.0	184.5	202.9	230.6	258.2	276.7	295.1	322.8	350.5	368.9	415.0	461.1	507.2	553.4	645.6
1200	94.2	103.6	113.0	131.9	150.7	169.5	188.4	207.2	235.5	263.7	282.6	301.4	329.7	357.9	376.8	423.8	470.9	518.0	565.1	659.3
1225	96.2	105.8	115.4	134.6	153.8	173.1	192.3	211.5	240.4	269.2	288.5	307.7	336.5	365.4	384.6	432.7	480.8	528.8	576.9	673.1
1250	98.1	107.9	117.7	137.4	157.0	176.6	196.2	215.8	245.3	274.7	294.3	314.0	343.4	372.8	392.5	441.5	490.6	539.6	588.7	686.8
1275	100.1	110.1	120.1	140.1	160.1	180.1	200.1	220.2	250.2	280.2	300.2	320.2	350.3	380.3	400.3	450.3	500.4	550.4	600.4	700.5
1300	102.0	112.2	122.4	142.9	163.3	183.7	204.1	224.5	255.1	285.7	306.1	326.5	357.1	387.7	408.1	459.2	510.2	561.2	612.2	714.3
1325	104.0	114.4	124.8	145.6	166.4	187.2	208.0	228.8	260.0	291.2	312.0	332.8	364.0	395.2	416.0	468.0	520.0	572.0	624.0	728.0
1350	106.0	116.6	127.2	148.3	169.5	190.7	211.9	233.1	264.9	296.7	317.9	339.1	370.9	402.7	423.8	476.8	529.8	582.8	635.8	741.7
1375	107.9	118.7	129.5	151.1	172.7	194.3	215.8	237.4	269.8	302.2	323.8	345.4	377.7	410.1	431.7	485.7	539.6	593.6	647.5	755.5
1400	109.9	120.9	131.9	153.8	175.8	197.8	219.8	241.7	274.7	307.7	329.7	351.6	384.6	417.6	439.5	494.5	549.4	604.4	659.3	769.2
1425	111.8	123.0	134.2	156.6	179.0	201.3	223.7	246.1	279.6	313.2	335.5	357.9	391.5	425.0	447.4	503.3	559.2	615.2	671.1	782.9
1450	113.8	125.2	136.6	159.3	182.1	204.9	227.6	250.4	284.5	318.7	341.4	364.2	398.3	432.5	455.2	512.1	569.1	626.0	682.9	796.7
1475	115.8	127.4	138.9	162.1	185.2	208.4	231.5	254.7	289.4	324.2	347.3	370.5	405.2	439.9	463.1	521.0	578.9	636.8	694.6	810.4
1500	117.7	129.5	141.3	164.8	188.4	211.9	235.5	259.0	294.3	329.7	353.2	376.8	412.1	447.4	470.9	529.8	588.7	647.5	706.4	824.1
1525	119.7	131.7	143.6	167.6	191.5	215.5	239.4	263.3	299.2	335.2	359.1	383.0	418.9	454.8	478.8	538.6	598.5	658.3	718.2	837.9
1550	121.7	133.8	146.0	170.3	194.7	219.0	243.3	267.7	304.1	340.6	365.0	389.3	425.8	462.3	486.6	547.5	608.3	669.1	730.0	851.6
1575	123.6	136.0	148.3	173.1	197.8	222.5	247.2	272.0	309.1	346.1	370.9	395.6	432.7	469.8	494.5	556.3	618.1	679.9	741.7	865.4
1600	125.6	138.1	150.7	175.8	200.9	226.1	251.2	276.3	314.0	351.6	376.8	401.9	439.5	477.2	502.3	565.1	627.9	690.7	753.5	879.1
1625	127.5	140.3	153.1	178.6	204.1	229.6	255.1	280.6	318.9	357.1	382.6	408.1	446.4	484.7	510.2	574.0	637.7	701.5	765.3	892.8
1650	129.5	142.5	155.4	181.3	207.2	233.1	259.0	284.9	323.8	362.6	388.5	414.4	453.3	492.1	518.0	582.8	647.5	712.3	777.1	906.6
1675	131.5	144.6	157.8	184.1	210.4	236.6	262.9	289.2	328.7	368.1	394.4	420.7	460.1	499.6	525.9	591.6	657.4	723.1	788.8	920.3
1700	133.4	146.8	160.1	186.8	213.5	240.2	266.9	293.6	333.6	373.6	400.3	427.0	467.0	507.0	533.7	600.4	667.2	733.9	800.6	934.0
1725	135.4	148.9	162.5	189.6	216.6	243.7	270.8	297.9	338.5	379.1	406.2	433.3	473,9	514.5	541.6	609.3	677.0	744.7	812.4	947.8
1750	137.4	151.1	164.8	192.3	219.8	247.2	274.7	302.2	343.4	384.6	412.1	439.5	480.8	522.0	549.4	618.1	686.8	755.5	824.1	961.5
1775	139.3	153.3	167.2	195.0	222.9	250.8	278.6	306.5	348.3	390.1	418.0	445.8	487.6	529.4	557.3	626.9	696.6	766.3	835.9	975.2
1800	141.3	155.4	169.5	197.8	226.1	254.3	282.6	310.8	353.2	395.6	423.8	452.1	494.5	536.9	565.1	635.8	706.4	777.1	847.7	989.0

FLAT ROLLED STEEL, Weight per Linear Meter, Kilograms